NOV 23 1956

PRESTRESSED CONCRETE INSTITUTE

Vol. 1 Sept. 1956 No. 2

NOV 27 1956

JOURNAL

OF PRESTRESSED CONCRETE INSTITUTE

IN THIS ISSUE

Proceedings — 2nd Annual Convention

May 16-17-18, 1956

Prestressed Concrete Research at the University of Florida ..... R. W. Kluge

Panel Discussion ..... Application

Panel Discussion ..... Production and Inspection

Panel Discussion ..... Design

Panel Discussion ..... Materials and Techniques

Panel Discussion ..... Specifications

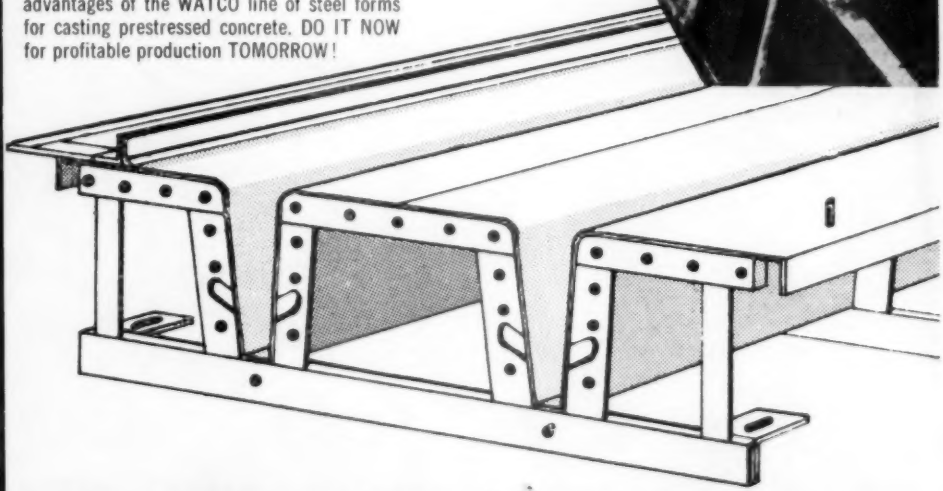
Current Scarcity of Engineers ..... Walter L. Lowry, Jr.

WATCO

...a complete line of first quality steel forms to meet every requirement for the efficient mass production of standardized prestressed concrete units.

WATCO Double Tee forms are of two-piece construction welded at one point only — the underside of the flange—for more attractive finished pieces. WATCO bridge girder forms are offered in standard depths and in multiples of 10 feet to meet State Road Dept. specifications.

Investigate the versatility and time saving advantages of the WATCO line of steel forms for casting prestressed concrete. DO IT NOW for profitable production TOMORROW!



PLANT CITY *Welding & Tank Company*

STEEL **watco** FORMS

P. O. BOX 1308 PLANT CITY, FLORIDA

NOV 23 1956

TA680
P83

JOURNAL

Vol. 1 September 1956 No. 2

OFFICERS & DIRECTORS
1956-1957

J. Ashton Gray
President

Ben C. Gerwick, Jr.
Vice President

Douglas Cone
Secretary-Treasurer

DIRECTORS

Peter Verna, Jr.

Harold A. Price

Werner Rosch

Carroll Strohm, Jr.

Arthur C. Bruce

Dr. T. Y. Lin

Charles C. Zollman

George W. Ford

From the editor . . .

The panel discussions appearing in this issue of the **JOURNAL** were recorded by court reporters during the Second Annual Convention of the Prestressed Concrete Institute as they were delivered by the respective panel members. During the course of each discussion, the panel members covered certain phases of the prestressed concrete industry. This was followed by a question and answer period in which the registrants participated.

In editing the remarks made during the panel discussions and the question period, every effort was made to keep the meaning of the ideas as they were recorded.

Each panel member was given the opportunity to review the transcript and correct obvious errors, provided such editing was completed before the publication deadline. In some cases it was necessary for the Editor to assume the responsibility of making changes in the context. If any of the thoughts or views were misquoted or their meaning changed by the editing, the Editor regrets the error.

The **JOURNAL** is published quarterly by the Prestressed Concrete Institute, Inc. **TECHNICAL EDITOR:** Dr. A. M. Ozell, University of Florida, Gainesville, Florida. **ADVERTISING, SUBSCRIPTION** and **PUBLICATION OFFICE:** 3132 N. E. 9th Street, Fort Lauderdale, Florida. **DISTRIBUTION:** Available without cost to all members of the Prestressed Concrete Institute. Additional copies offered to members at $1.00 per copy. **SUBSCRIPTIONS:** Subscriptions are solicited only from persons with an identifiable commercial or professional interest in building design and construction. Single copies: $2.00. Subscription rates: $6.00 per year.

Editorial contributions to the **JOURNAL** are welcomed, but publication cannot be guaranteed. All manuscripts should be submitted in duplicate and will be reviewed and approved by the Technical Editor prior to publication. Advertisements of products, services and materials allied to the prestressed concrete industry are welcomed in the **JOURNAL**. Address all inquiries concerning advertising rates and mechanical specifications to the Publication Office.

What is P.C.I.?

By J. Rex Farrior, Jr.

Legal Counselor for P. C. I.
Tampa, Florida

P.C.I. is legally known as Prestressed Concrete Institute, Inc., a corporation not for profit organized and existing under the statutes of Florida, Chapter 617. The Institute is legally recognized as a scientific, charitable and benevolent association.

The original charter was approved by the Circuit Court at Tampa, Florida, on June 18, 1954. Officially, the object of the Institute is the formation of an association of manufacturers of prestressed concrete and of persons and companies in allied or related industries for the mutual exchange of information and ideas; and generally to do anything beneficial for its members and the prestressing business which does not violate the laws of Florida or the United States.

The charter provides for six classes of members: Active, for prestressed manufacturers; Associate, for related businesses; Professional, for architects and engineers; Junior, for architects and engineers in training; Student, for students enrolled in accredited architectural and engineering colleges, and Honorary, for such persons as the Board of Directors may wish to honor.

From a tax standpoint, the Institute enjoys an enviable position. Being a valid non-profit corporation, it is not required to pay any tax on its receipts, and therefore 100 per cent of its income can be devoted to research and other useful projects.

Fortunately, also, each member's dues and contributions are completely deductible by the member for income tax purposes.

Both tax factors therefore guarantee to the Institute's members more than full value for each dollar invested.

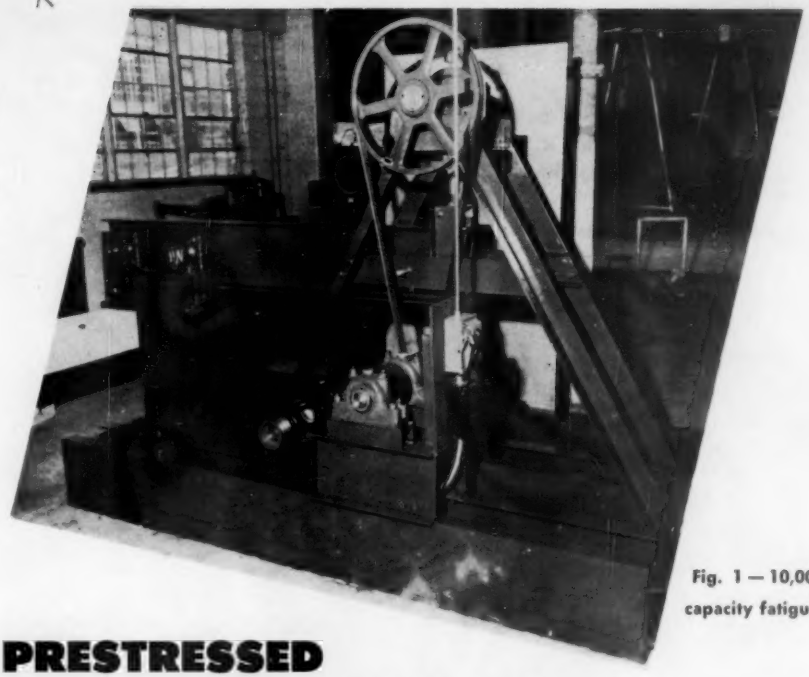


Fig. 1 — 10,000 pound capacity fatigue machine.

PRESTRESSED Concrete RESEARCH at the University of Florida

by R. W. KLUGE

Head Professor, Civil Engineering Department

Charles Kettering, Director of Research for General Motors, once defined research as ". . . an organized method of finding out what you are going to do when you can't keep on doing what you are doing now." Although expressed in a somewhat facetious tone this definition nevertheless makes considerable sense. Certainly we can't keep on doing what we were doing yesterday in prestressed concrete if progress is to continue. The University of Florida in cooperation with the Prestressed Concrete Institute and the Florida State Road Department is therefore currently engaged, to express it in a similar vein, in an organized attempt to determine what we can or can't do in this rapidly developing technique of prestressing.

It is the purpose of this paper to describe briefly the several experimental studies in prestressed concrete now being conducted at the University and briefly present some of the significant results of these studies.

It may be pertinent first to point out some of the physical facilities of the structural laboratory where this work is being done. It includes two beam fatigue testing machines, designed and constructed by Dr. A. M. Ozell of the Civil Engineering staff, and six standard static load testing machines ranging in capacity from 50,000 lb. to 400,000 lb. The smaller of the fatigue machines is of 10,000-lb. capacity, shown in Fig.1. The larger machine has a capacity of 50,000 lb. and is shown in Fig. 2. Adjuncts to the laboratory include a concrete and cement laboratory containing two types of concrete mixers, scales and other equipment necessary for casting concrete, as well as a large temperature and humidity controlled curing room which can be used for either high temperature steam curing or the standard 70 degree, 100 percent humidity curing.

In the production of good quality prestressed pretensioned structural units, two factors of considerable importance are: the method and the duration of curing. Two common curing methods currently used in the industry are the standard wet burlap cure or its equivalent, and steam or hot water curing. Duration of cure is obviously

This paper was presented at the Second Annual Convention of the Prestressed Concrete Institute, May 16, 17, 18, 1956.

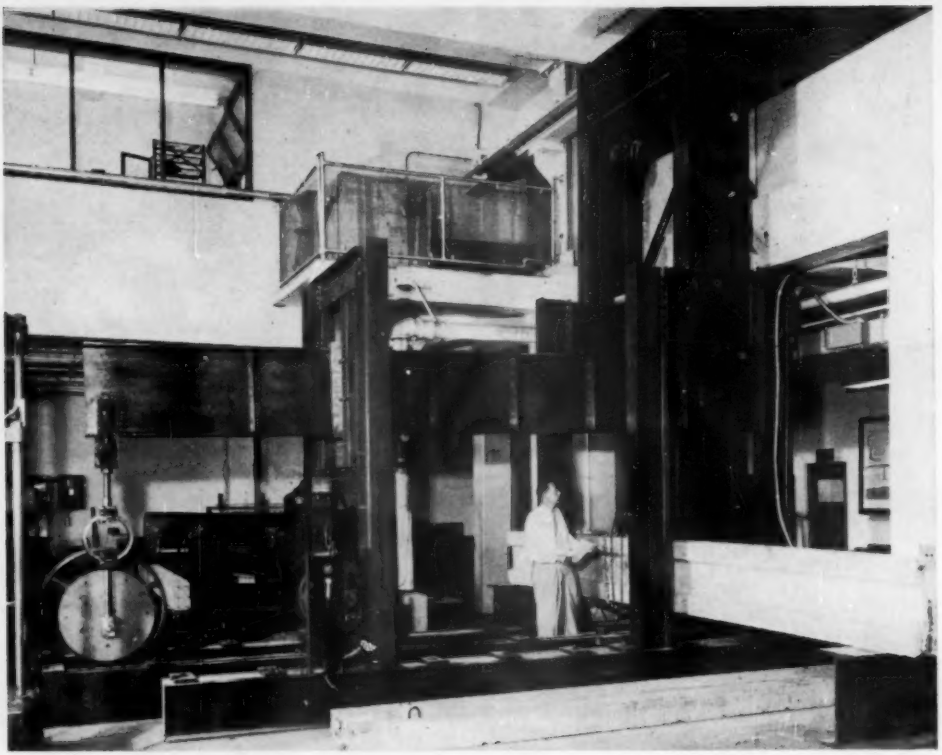


Fig. 2 — 50,000 pound capacity fatigue machine.

dependent upon the method of curing employed. From the standpoint of overhead costs, curing should be as short as possible consistent with the production of quality units. The rapid gain in strength of concrete is not so much a problem as the rapidity with which its elastic properties are developed, an important factor in camber control. There are also other questions related to the method and duration of curing such as the bond resistance of the strands. One of the investigations conducted at the University of Florida had as its purpose the gathering of some information related to the aforementioned problems, specifically the effect of normal and high temperature curing on the gain in strength and elastic properties of concrete normally developing about 3500 psi in 7 days. Also studied was the effect of method and duration of curing on the bond resistance of 7/16-inch strands.

The rate of gain in strength in periods from 1 to 7 days is shown in Fig. 3, the upper curve is for steam curing and the lower curve for standard moist cure. Similar data are shown for rate of gain in elastic modulus in Fig. 4. Again the upper curve is for steam curing. Fig. 5 combines the information given in the last two illustrations by comparing the ratio of elastic modulus to compressive strength for each day of cure. It may be observed that the elastic modulus for moist cure concrete is somewhat higher in proportion to its strength for early ages, but the steam cured concrete soon catches up with it. This does not present a completely fair picture of the situation since at one day, for example, the moist cured concrete has a strength of only 600 psi whereas the steam cured concrete is about 3 times that value or about 1800 psi. Fig. 6 makes this comparison on a strength for strength basis. It may be observed here that for strengths less than 3000 psi the moist cured concrete shows a higher elastic modulus, but for strength greater than 3000 psi, the steam cured concrete develops a higher modulus. It should be noted that this data applies only to a concrete having a cement factor of 6½ bags, type I cement with no admixtures. Departures from these conditions may alter the relationship one way or another.

The bond resistance of strands embedded in 6-in. by 6-in. by 6-ft. beams loaded at midspan was measured in terms of the magnitude of the load causing end slip of the strand which was exposed at each end of the beam. The maximum values obtained seemed to be a function of the concrete strength only. There appeared to be no significant differences between the steam and moist cured specimens. In general, below a strength of 3000 psi, the maximum load producing end-

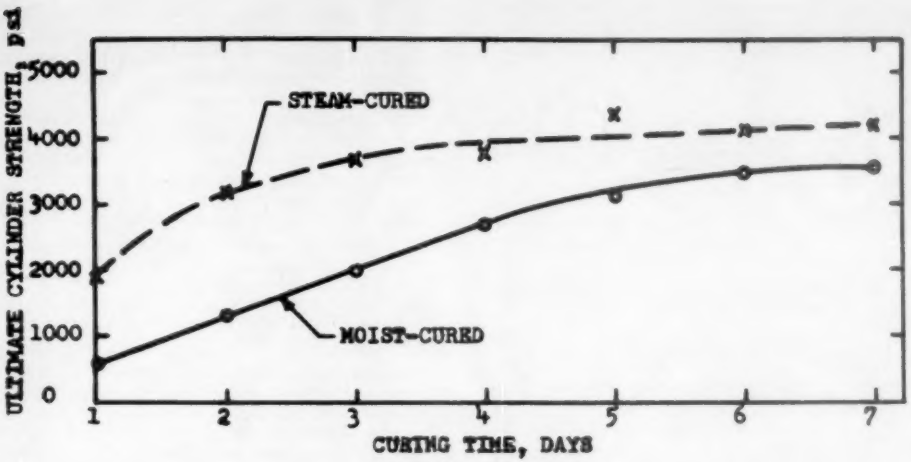


Fig. 3 — Rate of gain in strength.

slip decreased rather rapidly. One significant observation which came about unintentionally was that a sudden release of strands from the tensioning beds seriously affected the load carrying capacity of the beams. It is planned to extend this study of bond resistance to full sized beams under casting yard conditions some time this summer.

The behavior of prestressed concrete beams subjected to repetitive loading is of particular concern to bridge engineers. Although some fatigue tests have been conducted in Europe and this country there is still much to be learned about the fatigue life of prestressed members under various conditions of load and stress. One of the many questions concerns the bond resistance of large strands in pretensioned members. When one considers that anchorage of these strands is by bond developed in the end two or three feet of the beam and that a rather high bond stress occurs at the entrance of the strand into the concrete under prestress itself, there is ample reason

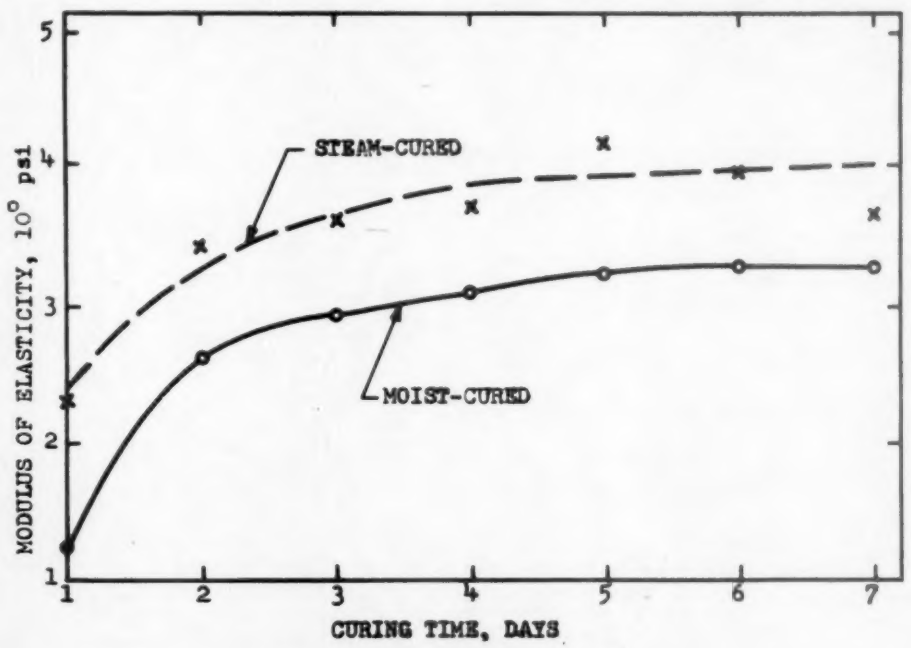


Fig. 4 — Rate of gain in elastic modulus.

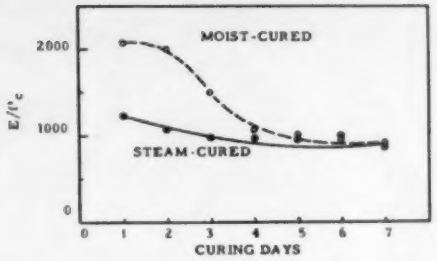


Fig. 5 — Ratio of elastic modulus to compressive strength.

to question its integrity under repeated applications of live load which adds to that bond stress.

The "Criteria for Prestressed Concrete Bridges" specifies the maximum strand size of 7-wire strands to be limited to ⅜ inch. In order to study the feasibility of using 7/16 inch strands from the standpoint of repetitive loads, a series of beams were tested in fatigue in which 7/16 inch strands were used.

Although the fatigue resistance of bond was uppermost in the planning of the test, it was soon evident that a load placed in such a position and of such magnitude to appreciably increase the bond stress, would, if moved to the center of the beam cause cracks to develop in the concrete under repeated loads. The cracks form serious stress raisers in the strands thus rendering them vulnerable to fatigue.

The study therefore developed into one of determining the fatigue life of the beam governed by the tension in the strand. Seven beams 6" x 8" on a span of 19' with 2-7/16" strands were tested under repeated loads until failure occurred in the strand, which meant the rupture of one or more of the wires in the strand. Failure of the wire was manifest by a sudden change in the load-deflection characteristic of the beam. The actual break was verified in each instance by a visual examination of the strand by breaking the concrete away in the region of the major cracks. Fig. 7 shows a typical crack pattern prior to failure. Fig. 8 shows a load-cycle diagram. The diagram indicates the number of repetitions of various magnitudes of load the beam was able to withstand before failure. It also indicates that for a load corresponding to 1.8 times the design load, the beam would withstand perhaps an infinite number of repetitions of this load. The value plotted to the far right contains an arrow indicating that failure did not occur at 3,000,000 cycles and it is presumed would not occur regardless of the number of load repetitions. It seems evident then that the use of 7/16-inch seven wire strands in pretensioned beams of these or similar dimensions and loading condition is feasible. Note, however, that although the fatigue strength of this beam is 1.8 times the design load, 2.0 times the design load caused strand failure at 940,000 cycles.

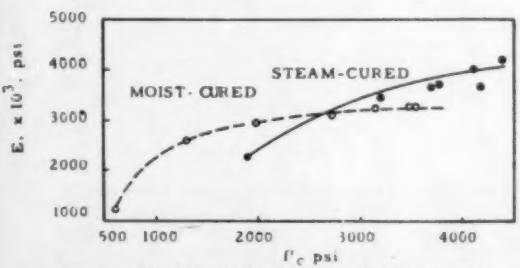


Fig. 6 — Elastic modulus versus cylinder strength.

Such tests are obviously time consuming and expensive, but are necessary if larger strands are to be safely used in pretensioning of members subjected to repeated loads. A similar series of tests are planned for ½-inch strands, not only with the load applied at the center of the beam but also applied near one end in order to induce bond failure as the primary type failure.

Another time consuming study although perhaps not nearly as expensive is the matter of determining stress loss in the reinforcement of prestressed beams and girders due to plastic flow and shrinkage of the concrete. There is considerable data available in the literature, based on laboratory observations, but very little data has been obtained from field observations under service conditions. The only known field studies in this country concerned with this problem were those conducted by the engineering personnel assigned to the Skyway Bridge Construction Project. There is reason to believe and some data to substantiate this belief that the effects of plastic flow and certainly of shrinkage are not as great as laboratory tests have indicated, when prestressed members are exposed to weather and particularly where the exposure is over water areas. In view of the limited data available, the University of Florida in cooperation with the Florida State Road Department initiated such a study in 1953 and now have under observation 6 girders in the Skyway Bridge, 4 girders in the recently completed Gandy Bridge, 2 girders in the Hardaway Construction Yard at Port Tampa, and 2 girders, seen in Fig. 9, located in the structural laboratory at the University of Florida. These girders are approximately 50 feet long and about 3½ feet deep, post-tensioned and pressure grouted.

Observations on the Skyway Bridge have been taken over a sufficiently long period to yield some information. Fig. 10 shows the average change in strain of the six girders or a total of 12 bars over a period of about two and one-half years. As is to be expected, the change is very rapid at an early age. Adding the slab to the girders brought the stress almost back to the initial prestress after which there was loss again for a period of about 60 days. It is not quite clear why

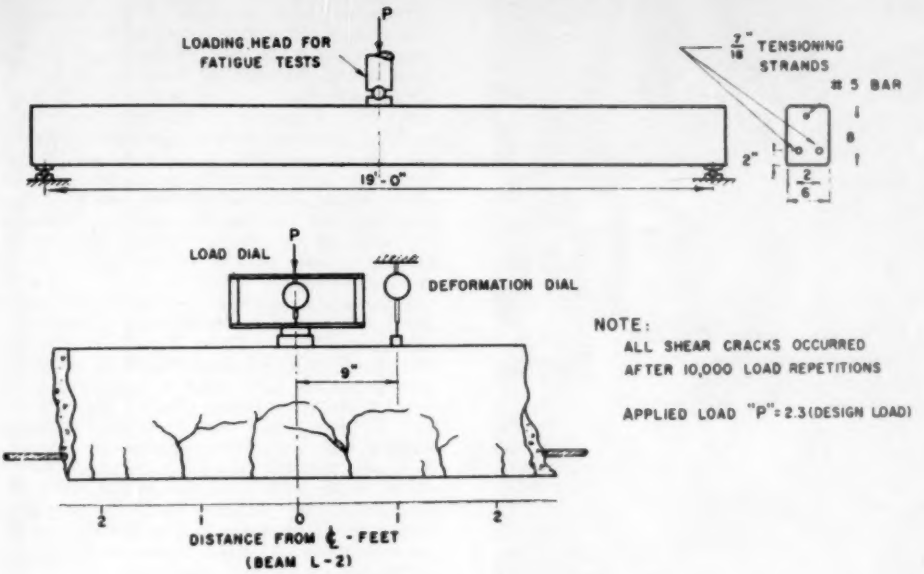


Fig. 7 — Typical crack pattern prior to failure.

the strain again increased during the next 100 days unless it might have been due to shrinkage of the slab or due to the effects of seasonal temperature changes on the bridge structure. Another increase in strain may be observed approximately 460 days later, indicating the latter possibility. Since the girders in this span are not free sliding at either end, seasonal temperature changes could reasonably be expected to affect the stress in both the concrete and reinforcement. It was because of this behavior that control girders outside of the Gandy Bridge, the two stored in the Hardaway Construction Yard, as well as the two in the laboratory, are being concurrently observed with those in the bridge. Observations on the Gandy Bridge have not progressed far enough as yet to provide much information. Studies of this nature are also planned for pretensioned girders which the State Road Department expects or perhaps are now using in smaller bridges.

The feasibility of prestressing long concrete columns was also investigated at the University of Florida. It seems evident that prestressing a short compression member presents no advantage, in fact, it can be

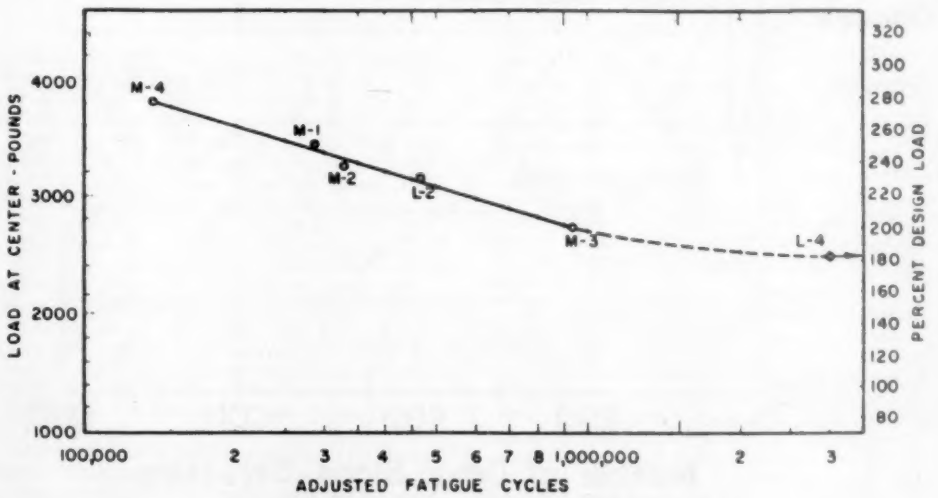


Fig. 8 — Diagram showing load versus cycle of failure.

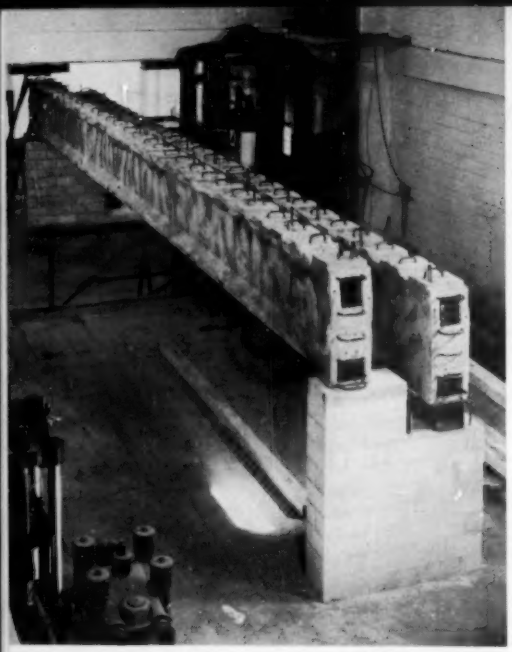


Fig. 9 — Girders used for shrinkage and plastic flow studies.

detrimental to its strength. It was believed, however, that for long columns with an L/d ratio, which is the ratio of the length to the least lateral dimensions, from say 20 to 30, prestressing may be advantageous in that the effect of accidental eccentricities which lead to buckling failures in such columns may be compensated to some extent by the tension in symmetrically placed strands. In other words, that prestressing would delay buckling and increase the ultimate strength by virtue of the greater bending stiffness of an uncracked section as compared to a cracked section.

Forty-seven columns, of which forty-one were prestressed, were tested. They were all nominally 6" x 6" or 8" x 8" in section and varied in length from 6 ft. 8 in. to 16 ft. thus providing L/d ratios of from about 10 to 32. The prestressing strands were ⅜-in. and 7/16-in., and the number of such strands was 4 and 8 symmetrically placed. Concrete strength varied from about 6000 psi to 8000 psi. Since the strand were all stressed to their working loads the prestress varied in direct ratio to the percentage of prestressing steel. Fig. 11 shows the manner of failure for tied columns.

Unlike conventionally reinforced concrete columns the steel in prestressed columns adds nothing to the strength of the column as a compression resisting element, therefore the ultimate strength is best expressed as the ultimate load divided by the net cross sectional area of concrete. In order to make comparisons between columns having different concrete strengths, a ratio of the

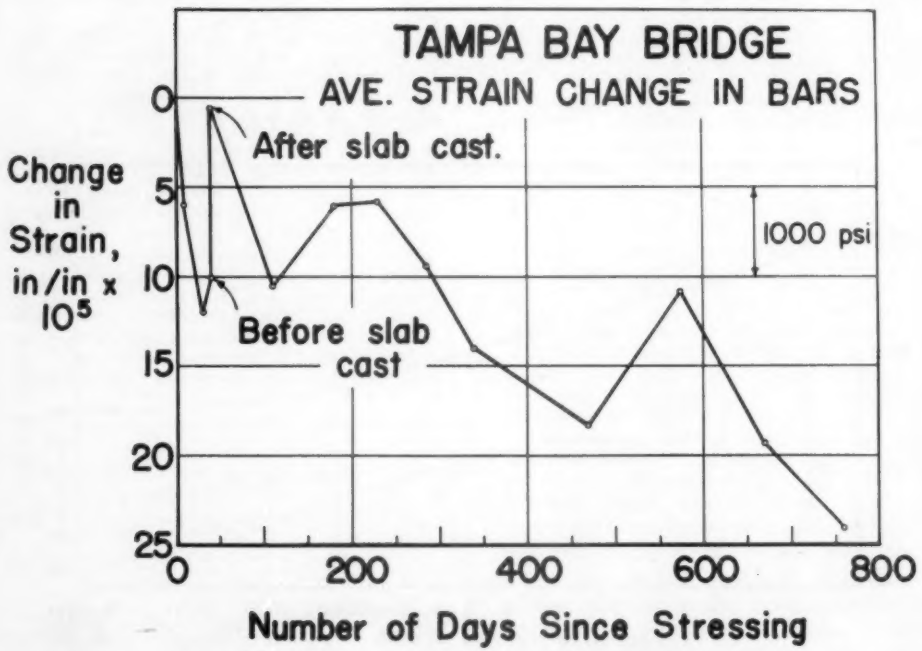


Fig. 10 — Average change in strain on Tampa Bay Bridge girders.

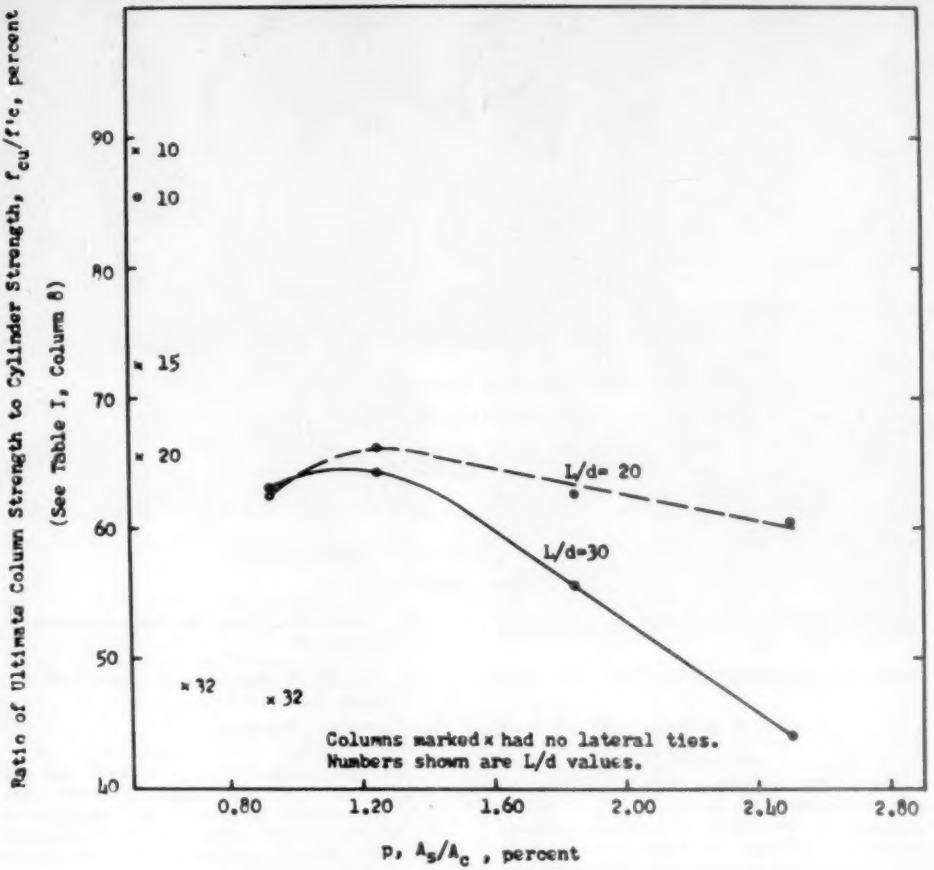


Fig. 11 — Capacity of prestressed concrete columns as effected by "p".

column strength to the cylinder strength is therefore necessary. Fig. 11 shows this ratio as ordinates and the percentage of prestressing steel as absissa with two curves expressing the relationship for the two parameters, L/d of 20 and of 30. The curves indicate an optimum value at about 1.25 percent steel which corresponds to a precompression of about 25 percent of the strength of the concrete. Precompression, of greater or smaller magnitude, reduces the strength of the column. This marked reduction in strength for the very long column (L/d of 30 as compared with L/d of 20) was contrary to expectations. Furthermore, the strength of a prestressed long column with an L/d of 30 and L/d of 20 differed little from the strength of conventionally reinforced columns of the same L/d ratios with comparable steel percentages. It would therefore appear from these results that there is no advantage in prestressing long columns axially loaded except for handling as a precast member. It might be pointed out that experience with another type of long compression member, the foundation pile, has been extremely satisfactory at least from the standpoint of handling and driving.

Another of the projects conducted in our laboratory, Composite Beams, is merely mentioned here since it has been reported in detail in the first issue of the new Journal of the Prestressed Concrete Institute, which I am sure you all have.

The report just presented, of course, barely touches upon the many problems in prestressed concrete requiring study. It represents a contribution from the structural laboratory at the University of Florida, accomplished for the most part, in less than a year. In another year it is hoped that considerably more will be added to our knowledge of the behavior of prestressed concrete. Acknowledgement for the material reported upon is given to Dr. A. M. Ozell.

Second Annual Prestressed Concrete Institute Convention

PANEL DISCUSSION

APPLICATION

CHAIRMAN

H. H. Edwards, Leap Concrete, Inc., Lakeland, Fla.

PANEL MEMBERS

Eugene Smith, Freyssinet Company, New York City

E. J. Ruble, Association of American Railroads, Chicago, Ill.

Ross Bryan, Consulting Engineer, Nashville, Tenn.

J. C. Rundlett, Bridge Engineer, Dept. Public Works, Boston, Mass.

R. C. Dorland, Southwest Structural Concrete Corp., San Diego, Cal.

MR. RUBLE:

Before presenting this discussion, it would appear desirable to acquaint you with the organization known as the Association of American Railroads so that you can fully understand how the railroads conduct their joint research.

The A.A.R. is an organization of about 200 railroads in the United States, Canada and Mexico, with a total mileage of about 225,000 miles. The railroads as a whole form a vast laboratory for research under actual operating conditions, and it is the policy of our staff to take full advantage of this laboratory for the final evaluation of our work.

The research staff has a program of investigating all phases of railroad engineering. However, my work deals principally with the 192,000 dynamically loaded railroad bridges of timber, concrete and steel. The railroad bridge engineers realize that considerable data is available on prestressed concrete for buildings and highway bridges; however, there is very little data available for railroad bridges. And they felt that we should secure some data on railroad bridges.

For example, the bending moments and shears in a railroad bridge are about six times greater than those in a highway bridge. And of course, this involves quite a problem. Preliminary calculations for a prestressed railroad bridge indicated that we would have to use one-half inch strands, a size not commonly used at this time.

To secure data on a typical railroad bridge slab, we made arrangements with the Prestressed Concrete of Colorado to build three such slabs, one for laboratory testing and two for installation in a railroad bridge. We also made arrangements with the Bureau of Reclamation of Denver for the use of their 5,000,000 pound testing machine for the testing of one slab. This testing machine was selected principally on account of its large capacity and also on account of the 10-foot clear distance between the screws. Then, we made arrangements with the Burlington Railroad for the installation of two slabs provided that the test slab proved satisfactory.

Fig. 1 is a view of the casting yard of the Prestressed Concrete of Colorado in Denver. You will note in the bottom of the yard that we have 61 one-half inch strands. The hydraulic jacks are housed in the building at the end. To stress these strands up to 140,000 pounds per square inch, a total load of 1,300,000 pounds was required.

Fig. 2 shows the jacking arrangement. In general, about 20 strands were stressed at one time; they were then locked in position and 20 more strands were stressed and so forth until all the strands were stressed.

I believe that it would be of interest to state that we took strain gauge readings on the strands after the cables were cut,

Ruble Fig. 1 — View of casting yard of Prestressed Concrete of Colorado in Denver.

and we found that the half-inch strands developed full bond within about the first six inches of their length.

Fig. 3 shows a detail of a typical railroad slab. Two slabs were put in service. The slab is 19'11½" long, 6'2¾" wide and 1'6" deep.

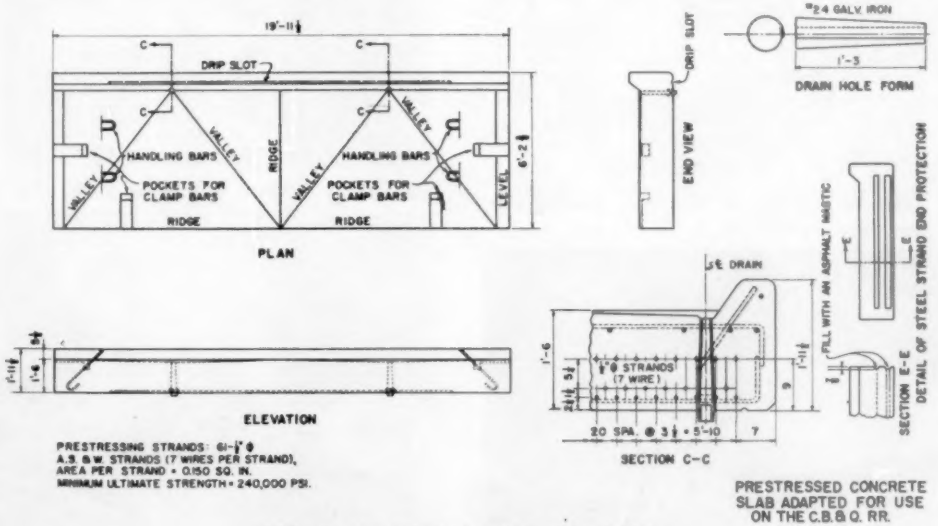
Now, the regular slab would be about four inches deeper than this prestressed slab. And, we used about two slabs for each span. You will note the ridges and the valleys that are on this slab which are for drainage purposes, and also note the curve which is to retain the ballast. You will also note a detail at the end of the slab is an insert which was used to protect the end of the steel before it went into service.

Fig. 4 is the view of the five million pound testing machine at the Bureau of Reclamation. This slab is already in position for testing. You can note that the center of the testing machine head is slightly eccentric with the slab. This was done purposely as the actual load from the rail is about 2'6" from the center line of the track, consequently, we put the center line of the testing machine head 2'6" from the inside edge of the slab.

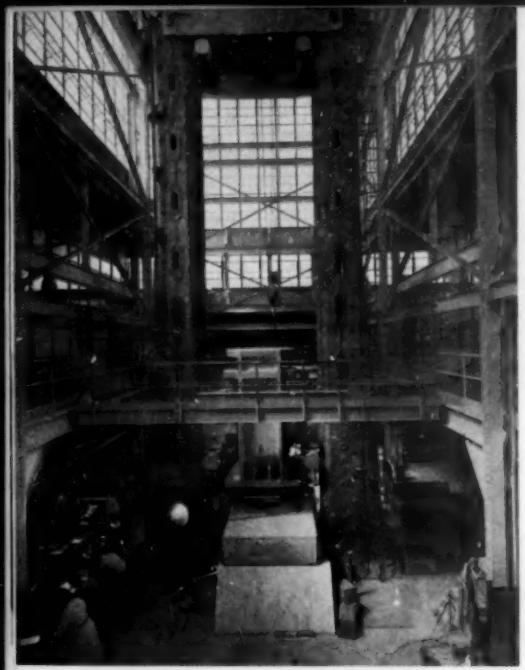
Ruble Fig. 2 — Jacking arrangement at casting bed.

At the lower left hand corner, you will note the instruments that were used to record continuous strains.

Fig. 5 is a detail of the test setup. The reaction at the end was carried on rubber fabric pads while the load from the test machine was applied through rollers which were five feet apart.



Ruble Fig. 3 — Detail of typical railroad slab.

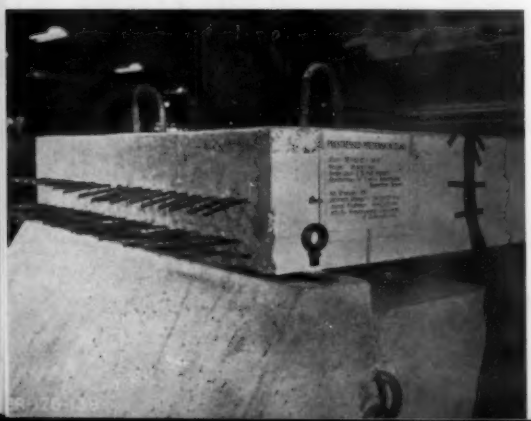


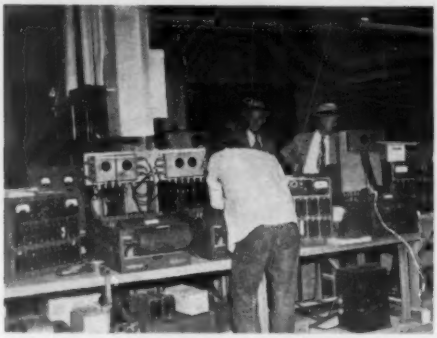
Ruble Fig. 4 — 5 million pound testing machine at Bureau of Reclamation.

Fig. 6 shows some of the strain-gauge equipment. We used three 12 limit oscillographs to record the continuous strains in the slab up to failure. These strains were recorded on film. This view shows the various oscillographs, amplifiers and power units that are used in the testing. The results of the test indicated that the slab was carrying about 3.2 times the design load with a deflection of about two and a quarter inches at an ultimate load of about 550,000 pounds.

Fig. 7 is a view of the construction of the bridge. The prestressed slabs form an approach span to the steel span and can be seen just to the left of the steel span. This bridge is built in a line change on the Burlington Railroad on the Kansas City-Chicago line.

Ruble Fig. 5 — View of test set-up showing end of slab.





Ruble Fig. 6 — Section of the electronic strain-gauge test instruments.

Fig. 8 is a detail of the method of setting the slabs. First, a three-quarter-inch bed of mortar was spread on the cap, and then a 12-inch zinc plate was placed on the mortar. The slab was then seated on the zinc plate. Now, this is not necessarily a recommendation for all railroads to use, but it is a method that the Burlington Railroad has used for years. We feel that a rubber fabric pad would be more satisfactory.

Fig. 9 is a closeup view of the slab being seated on the zinc plate. You will note that the recess at the end of the slab has now been filled in. An interesting note about these slabs is that they were shipped out to the bridge, and we got a wire from the master carpenter on the job that the concrete was no good in these slabs. He said that you can take a hammer and hit it and it would ring. Well, actually, we had concrete of an ultimate strength between 8,000 to 9,000 psi.

Upon completion of the line change, when the bridge was in operation, we went down and took some strain gauge readings on the bridge under moving loads from a slow speed of, say, five miles per hour, which is considered static, on up to a maximum speed of 80 or 90 miles per hour (Figs. 10 and 11).

Ruble Fig. 7 — Construction of the bridge showing crane placing prestressed concrete slab.

Ruble Fig. 8 — Detail of the method of setting slabs showing bearing pad.

Ruble Fig. 9 — Closeup of slab being placed on pier and being seated on the zinc plate.

Ruble Fig. 10 — Taking strain gauge readings on the bridge under loads.

Ruble Fig. 11 — Diesel locomotive on prestressed span.

Now, the impaction appeared to be somewhat smaller than we would find in a regular-designed slab. And, this is due principally to the elasticity of the slab. We have found that on other structures where the slabs are supported on piles, the impactions are considerably lower than they are in the slabs supported on solid foundations. However, we have not completed our analysis of the data. We hope to have a final report about this sometime next year.

CHAIRMAN EDWARDS:

Our next speaker will be Mr. Dorland from California. Mr. Dorland has done some very interesting work along the line of composite design. I am sure you will all be glad to hear what he has to say about their composite design work.

MR. DORLAND:

Our biggest problem in the San Diego area has been a sales problem. We find that if you can sell it cheaper than anyone else, you can sell your product. But, if it is a little bit more expensive, you have a difficult time selling your product on the merits of being a superior product. Consequently, almost all the work we have done has been on an alternate design basis. This presents a challenge to the engineer as well as to the plant that makes the product, to manufacture a good product at the lowest economical price so that you can actually get the job.

Most of our bids have been that we have provided our own design in competition with a standard design of wood or steel. The particular bridge that I am going to talk about was a very small bridge where we actually provided the design for a prestressed bridge in competition with a wood bridge.

The bridge was 165 feet long, consisting of six spans, and each span was around 28 feet. This was dictated by the base bid, and we were not able to change the spans. They were not the most economical for prestressed concrete. When the bids came in, wood was at $24,900.00 and the prestressed concrete at $28,500.00. The City elected to take the prestressed concrete bridge, even though its cost was slightly more, because of the low maintenance cost. It was a challenge to actually make a small bridge and pay off the forms and still make a profit on the job on a bridge of that nature. This was the first bridge that was constructed out of prestressed concrete in the San Diego area. And, the way we were able to effect economies was to make a composite section.

The bridge had channel sections that were 4½ feet wide, 28 inches deep and 28½ feet long. Rather than making expensive forms and side forms, we analyzed and found that it was much cheaper to make a composite section if the legs were produced on a flat bed so as to eliminate almost all of the forms. You just had two side forms and only one side form had to be operated. The

ribs were five inches thick, and were prestressed. The bridge was designed for H-20-S-16 loading.

The ribs were produced by an assembly line technique on a 200 foot bed. These were cast flat, and then two were set up in a vertical position and then flat soffit forms were placed in between the two ribs. This meant that we could re-use our soffit forms every single day. Steel extended from the legs over into the deck slab, and the steel for the deck was also placed before concrete was poured.

The day after the pouring of the deck slabs, we could lift the channel section out and stockpile it.

The ribs were made with 5,000 psi concrete, and the deck slab with only 3,000 psi concrete. The latter actually could have been 2,000 psi concrete, but for quality reasons, we went to a 3,000 psi concrete. By doing this, we could also control the camber of the finished product. That is, after you set your ribs up in your jigs, you could pour your deck to a uniform elevation and have essentially a flat unit.

All of these units were produced in the plant, and then hauled to the job site. The piles were previously driven and pile caps prepared by the general contractor.

There were six sets of channel forms set side by side forming the 28-foot width. The site was over water so that you could not get equipment into the center. So, we had the channels designed to carry the erection loads. And, the procedure was as follows: You would set the first channel, then the crane would walk over it, lift the next channel off the truck, and set it down.

We were very pleased with this erection procedure. We anticipated to have about three to four days erection time on the bridge. It went so fast that we actually erected the entire bridge in 14 hours with no stockpiling necessary at the job. The trucks would drive right on to the bridge and the crane would unload those while the truck would move right on off to pick up more channels back at the plant. We were very pleased with the way the bridge came out. And the City was pleased and subsequently they gave us about three or four more designs for prestressed concrete.

MR. RUNDLETT:

I am going to talk on the "Standardization of Prestressed Units."

In my recent papers on prestressing presented before the American Society of Civil Engineers and the Boston Society I have indicated the variety of sections used in precast prestressed concrete members for bridge work and have expressed the need for standardization of monolithic beams. Standardization is particularly important in prefabricated sections manufactured in established yards, and my few remarks are confined to precast units of this nature—those made in a plant and transported to the bridge sites. It is only through an assembly - line - standardized - product that economy can be realized; and consequently a greater use made of this new and increasingly popular construction material.

In the beginning it was only natural that individuals should develop their own ideas and it was through these diversified developments that we have progressed so far and so rapidly on short-span-precast-construction.

Ideas must be tried, discarded, revised and tried again, but there comes a time when it is essential for the future of the industry that designs be consolidated, when individual preferences must be given second place, and where uniformity must be established.

Although I have made no survey of the situation this year, on the basis of past observations it would seem that Florida is far outstripping the rest of the country in developing prestressing and in standardization. This is natural because you have here the ideal combination of a progressive Bridge Engineer, Bill Dean, and a climate ideally suited to the use of concrete.

Individual designs have been made by many engineers for beams of every conceivable shape and with various methods of tensioning. In the early stages of development, Massachusetts and Florida concentrated on post-tensioned I-sections, and the designers of the Garden State Parkway made use of pretensioned I-sections. We have channel-sections used on the Future Farmers of America Bridge in West Virginia, and octagonal sections on a bridge in Denver. The Atlantic Street Bridge in the District of Columbia made use of hollow square-beams, and the Hubbard Avenue Bridge in Connecticut contains beams that are T in shape. Other installations called for T-sections, with the top flange of the beam made a part of the slab, with the stems butted, while other T-sections were used with the stems separated and the space filled with concrete and prestressed laterally.

There seems to be no need of further searchings for other sections. Enough beams have been designed so that certain depths and shapes can be accepted and the designer required only to calculate the stressing units to fit the shape, the span, and the loading. Even the shape can be standardized if necessary, but I feel that this should still, in general, be left open to competitive systems.

At the moment, I would concentrate on three basic types: namely, the rectangular-section, the channel-section, and I-section, and I think I would discard the others from consideration, particularly those which require lateral prestressing in the field.

The general span-range for precast prestressed units is from 20 feet to about 70 feet, with the latter limit being governed by handling weight, length, and other practical considerations. Standardization, I think, should not at the present time go beyond the above limits.

For purpose of discussion, my suggestions on the above are as follows:

1. That for secondary roads, two basic depths of rectangular beams be established, 17 inches for spans of from 20 to 30 feet, and 21 inches for spans from 30 to 40 feet. If it is feasible that spans up to 50 feet in length be made of this type of beam, a third depth can be added. The beams, I believe, should all be pretensioned. The width can be either three or four feet, with my personal preference towards the latter, except that weight consideration may require a reduced width in spans over 40 feet. The width, too, will be determined by the standard width of roadway desired. In our case a four-foot beam fits well into our standard 26 and 30-foot secondary roadway widths and was adopted for the flood replacement bridges. Bridges of this type must be surfaced with bituminous concrete and, in our climate, membrane waterproofing must be applied to the top of the slab.
2. Similar considerations can be given to the use of the channel-section which will result in a reduction in weight.
3. That for primary roads, four basic depths and sections of a modified I-shape be established for a span range of from 25 to 70 feet; this type requiring a poured-in place slab. It seems quite unnecessary that a different beam depth be established for every five-foot change in span or for a change in loading, because the additional concrete and reinforcing steel called for on a deeper beam used on a shorter span is in the web, which is five or six inches in thickness.
4. That the sections established be of such width and shape that either pretensioning or post-tensioning can be used in the shorter spans, and that a combination of both can be used in the larger spans.
5. That spans of from 25 to 40 feet have a depth of 27 inches, that spans from 41 to 50 feet have a depth of 30 inches, that spans from 51 to 60 feet have a depth of 36 inches, and that spans from 61 to 70 feet have a depth of 40 inches.
6. That precast prestressed piles be standardized with sections 12, 14 and 18 inches, similar to those adopted in Florida.

The trend, of necessity, must be toward standardization. Pennsylvania has concentrated on the use of standard rectangular sections. Massachusetts has done likewise in its flood replacement program, and Connecticut is following suit. Florida has standardized on I-sections.

Now is the time for universal standardization, and your organization should, I believe, be the one to foster it.

MR. SMITH:

This afternoon I would just like to mention some possibilities that have not yet been very fully exploited here in the United States.

For the most part, all of our work has been confined almost exclusively to the standard flexual members, i.e., bridge beams, roof beams, floors of one form or another, whether it be double-T, I-beams, or whole slabs. We are pretty much restricted to designing only flexure mmebers.

Now, some engineers in the United States have modified this somewhat. They have come up with good competitive designs for particular applications; however, these particular designs have not been extended to general use. One simple example of this is the work that was done in New York State by a County Highway Superintendent who wanted cheap abutments for his bridges. Here is the situation where he had his men working like mad during the construction season and essentially, I would say, during the winter. He combined precast blocks with post-tensioning to produce his highway abutments. The blocks themselves would be cast during the winter season which was ordinarily dull, preformed holes would be supplied in them for the post-tensioning units and then when the spring construction season started, he would be able to deliver those blocks to the job site, place them on top of an existing footing and extend his abutments straight up to whatever elevation was required. The abutments, after the final elevation was reached and had the capping concrete placed on top, would be post-tensioned vertically to hold them together.

Now, this Highway Superintendent, Felix Ramsey, claims that by so doing, he was able to save as much as 50 per cent in cost and as much as 50 per cent in actual erection time. Now, it is conceivable, of course, that propositions like this may receive only very occasional application. However, it is interesting to note and would be worthwhile to keep in mind.

Another such application was the erection, several years ago, of a flexure member which was nothing more than several blocks of concrete prestressed in three directions. This was used for an eight ton forming hammer foundation. The prestressing done on that, frankly, looked like a mess with cables running in every direction. However, they got a very good job out of it finally, and are very happy with the operation of the hammer foundation and were apparently rather happy with the cost.

Again, I do not say that all hammer foundations are going to be prestressed from now on, but at least the special situation, if it should arise, can be solved by post-tensioning simply because of the introduction of fatigue stresses in such a structure.

Now, just recently in Toronto, the Toronto Cast Stone Company have erected bolstering trusses. That, as far as I know,

is the only one in the Western Hemisphere, although I am not certain of that. But, they were able to span up to 120 foot roof loads with those bolstering trusses. The prestressing in it is relatively small. The members themselves were precast and assembled in the field before lifting into place, and the total job was reasonable in price and was quickly assembled since most of the units were precast away from the job site.

Now, lift slabs in the United States, at least in certain regions are gaining in popularity. In other regions, they have not caught on too well at all. But, the obvious advantage of prestressing in a lift slab, of course, is pretty evident.

If we go into heavier units like Lake Ponchartrain down in Louisiana where they introduced the idea of precasting a complete slab, floating it into place and lifting it on to the pile caps. Those particular slabs weighed 180 tons. There probably will not be too many jobs the size of that in the United States, but the same proposition of working with complete slabs instead of working with lighter slabs is still open for future developments.

Floating units of one sort or another have been built here in the States and overseas. Up in New York Harbor, Pier 57 is completely founded on buoyant boxes. Most of the dead weight of the boxes are carried by their buoyancy. The remaining part of the dead weight is carried by the piles, which also, of course, take the superimposed light load. Those boxes were not prestressed, although a dense and crack-free concrete of high strength would be desirable in such structures.

In Europe and in South America they have done similar work with prestressed concrete. They have floating drydocks, floating barges, watertight structures of almost any kind that they have been able to prestress.

Along that line, the Harbor Unit in Le Havre has 1,700 ton precast units which were post-tensioned. These were for a wharf structure and they measured 82 feet square and were precast near a waterway and slid down on rails to a floating drydock which was also made of prestressed concrete. The floating drydock delivered them to the job site. The units themselves, by the air that would be trapped under them were largely buoyant, and were floated into their final positions.

Now, one other application, say of prestressed concrete, is accomplished by the use of flat jacks. There is no steel in the concrete, no actual tensioning of steel, but only the application of prestressing forces by external jacks. These have been used very successfully on arch bridges. The flat jacks are inserted into the crown of the arch, concrete poured for the arch itself and then after the concrete is hardened, by inflating those flat jacks in the crown, your prestressing force would be applied to the concrete and actually lift it off the false work. That has been used in a number of European bridges. It was used in the Caracas Bridge in South America, and in general, it has proved well worthwhile.

Now, the flat jack can go a little further. Of course, they can be used in any application where you have a strong abutment, and you want to put a direct compressive force on the concrete from the outside. They have also been used in prestressing rock formations when it was desired to maintain a certain pressure in soil for pipelines, etc. They have also been used for longitudinal prestressing of pipelines or tunnels. The flat jacks would be inserted in the pour, and when they are expanded they would force longitudinally the entire pipe and act as the prestress.

Now, these are ordinarily very cheap applications. The jacks themselves, of course, are expendable, very lightly made and very inexpensive. There is no steel to insert in the concrete, and in general, the economy is very considerable.

Another application for these flat jacks in Europe and North Africa was for runways. We have not done it here in the United States, but runways that were seven and a half inches deep with prestressed concrete in North Africa had to be 15 inches deep if they were made in regular reinforced concrete. The saving through prestressing was something like 50 per cent of the concrete material. These slabs would be transversely prestressed by the use of ordinary steel tendons. Every 1,000 feet or so along the length of these runways joints were provided in which flat jacks were inserted. After the concrete was poured and the transverse prestressing was applied, then the expansion of these flat jacks would put a longitudinal prestress throughout the length of the runway insuring crack free and much stronger concrete than a reinforced concrete construction.

Some of these may be applicable almost any day. Some of them may never be applicable. But, so far, in the United States prestressed concrete work was pretty well limited to flexural members. Although this is the most obvious application, and certainly the big moneymaker, we should not limit ourselves and at least give some consideration to the other applications.

MR. BRYAN:

I am going to talk to you about "Architecural Applications of Prestressed Concrete"—the problems of integrating prestressed concrete into architectural applications.

I am only going to tell you about the problems and let you work out the answers as best you can. It is really a rather touchy subject because we are dealing with a new material in which the architects as a group have not had any experience as yet. The engineers who are executing the architectural designs have not had experience. It is probably safe to say here that

most of the designs being executed economically are prepared by engineers who are specializing in prestressed concrete design.

The door to architectural acceptance is not always open to those of us who work in that field. The use of prestressed concrete by architects presents to them a number of unfamiliar problems, and there are problems that are not always easily answered. For instance, if you have several beams coming together on a column, you have to have a haunch of some sort, or a wide column to properly seat the beams. This is not always the case, but usually that is the suggested solution. And, that is not a good solution. The architect does not like it as it will not do anything for his building. The architect tries to keep the ceilings high, construction depths low and a haunched column simply does not fit into that picture.

The problem of making joints is an inherent problem to all precast work. Most of the economy comes from the fact that it is precast. And, in any precast product, the problem of assembly is a real problem. The joints are something with which the architects are not always happy, and unless you are able to hide it and make like there is no joint there, he does not like it. You have trouble making up standard widths and lengths for buildings with precast members. They have a way of growing in length and hanging over an inch into the brickwork, and that is expensive sometimes and very annoying to the architects.

Spandrel sections are also a real problem. Most of the modern architecture today tends to go to a very shallow spandrel construction. They like to have the windows go up to the ceiling. And, they prefer that the ceiling did not have any thickness at all, but they are willing to go along with seven or eight inches or something like that. And, actually, you will find a lot of resistance if you make that construction deeper than 12 inches.

In precast construction you have to set one beam on top of something or into something. And, in doing that you increase the framing depth. If you did not increase the framing depth, you increase the framing problem.

Camber is a problem to the producer, but to the architect it presents a number of problems. It plays havoc with his drainage pattern on occasions. He wants a flat roof. and if he has camber in his roof, it would throw the water to an area that he does not have drained. That requires additional fill. The additional fill requires an additional fascia thickness. Both of them cost more money, and the architect does not like the results.

The problem of inserts and openings for plumbing, lighting fixtures, hanging of ceilings, etc., can generally be solved, but the problem of locating openings is really a pretty tough problem in a complicated building where you have a lot of openings. The openings for stairwells, openings for vent pipes, openings for dumbwaiters, and that sort of thing, has to be accurately located. It has been our experience that sometimes it is better to cut those openings in the field. Usually, you can convince the architect that that is the way to do it. They feel that since you are making a precast product, you should really precast all those openings and set all those fixtures, and set all of the inserts, but somehow or other, they are not always in the right place when you get them out there, and a lot of the producers would like to do that work in the field. I am inclined to think that that is probably the place to do it.

The standard design, the standard building and standard product is, of course, a natural to a products man, and we are aiming for that. That trend produces a very cold and very still design and nine times out of ten there is nothing attractive about most of the standard concrete buildings that have gone up including the ones that we have done. It is because we are holding down economy, and it is because we do not know what is good looking lots of times.

Since the architect is working in an unfamiliar field, if you leave the design to him, he will make the project complicated; he will require you to do the things that you cannot execute without very expensive form work, and probably force you to build something that no one would ever build again.

So, you are stuck with a form if you follow an architect's pattern, and that means you are stuck with a high price and probably will not sell the product. So, those two diverging viewpoints of interest have got to be bent until they come a little bit closer together.

The problem is for those of us who are working as consultants in this field to work closer with the architects. We have been doing that to a great extent, and have made considerable headway. We find that the architect is more prone to help you if you go to him with a problem and make like he is helping you instead of going to him with a design and telling him that this is what he has to do. Lots of times if we talk in terms of dollars, he is more apt to listen. If you say that this is going to be more expensive should we do it this way, he will listen to you, but if you tell him it has to be done that way, he is apt to tell you that he does not like it and probably will not do it.

I think that we have to guide the architect into helping us produce an acceptable building. I do not think that any of us are happy with the structures that we are building. They are not pretty, with very few exceptions. They are not even well put together. Most of them are set up as posts and lintels and that is going back 50 years in the design of structures. Continuity has got to be integrated into them. And, we have got to provide continuity; we have got to provide it cheaply

and we have got to provide it well. The use of continuity will help us get rid of our haunches too.

I believe that the average architect likes prestressed concrete. He likes its permanence, and he likes what he gets out of it. But, unless it pleases him and pleases his client, he will be reluctant to accept it, and I think, the closer we work with him, the farther along we are going to get.

CHAIRMAN EDWARDS:

Your talk brings out the fact that prestressing being so new has brought a lot of problems with it, and a great many of those are in applications to architectural structures.

QUESTION:

I think that Mr. Ruble is certainly on the right track on these bridges. There is one thing that has struck a little chord with me, and I believe that you said your trend was away from post-tensioning the sections together, that is lateral post-tensioning?

MR. RUBLE:

That is my opinion.

QUESTION:

I see. We have had very good luck with it, and we are pleased with it. I guess we will have to compare notes. We think it is the most economical.

MR. RUBLE:

It is only my opinion in my particular little section, we have found the more you can concentrate that kind of work in the shop, the better off you are.

QUESTION:

Yes. I agree with that, but sometimes we put these things together, and we think of the cheapest way to get them together and also from the structural standpoint we think we can do a better job this way.

CHAIRMAN EDWARDS:

Would anyone else like to comment on the lateral post-tensioning of bridge work? It has been quite a controversial subject.

QUESTION:

Only to point out that in the bridges designed for the Illinois Turnpike, the same engineers who formerly designed post-tensioned diaphragms have designed precast diaphragms with welded plates to drop into place between the beams and be welded to similar plates cast in the beams.

CHAIRMAN EDWARDS:

That would be to take shear between adjacent members.

QUESTION:

It is no different than your channel between the two. The same engineer that designed that, previously used transerve post-tensioning on a group of bridges on a parkway. Although the results were structurally good, the problems involved were rather severe in terms of personnel and getting the job done.

MR. RUNDLETT:

I was particularly referring to the post-tensioning of slabs, not the diaphragms when I spoke of it. I mean, there has been instances where the entire slabs have been post-tensioned laterally in the field, and that was the particular point I was making.

QUESTION:

I would say that is even worse. I might also say that, as some of you may have seen that design, it utilizes a thin pretensioned two and a half inch thick plank between girder space of seven foot six inch, on which the topping is poured, inducing a composite and continuous design in both directions. There would be no form work on the job at all, just the structural concrete topping on the thin pretensioned plank resting on pretensioned I-beams.

CHAIRMAN EDWARDS:

Ross, would you like to say something along that line?

MR. BRYAN:

Yes. I would like to agree with this gentleman. I feel that the thinner the slab, the more need there is for transverse prestressing the deeper sections wider apart. Where there is any possibility of a slab being made more economically by a design that distributes the wheel loads, I think it is essential that the individual members be tied together in some manner. I do not think it has to be prestressed. They do have to be tied together, in my opinion. And, that tie has to be in the bottom of the slab where the tension exists to keep them from separating. I do not have any doubts whatsoever that the bridge would function and carry its design loads better; I do have a doubt that it would continue to function under excessive overloads and over a period of time without eventually wearing itself out at the joints.

CHAIRMAN EDWARDS:

Any further discussion.

QUESTION:

Mr. Ruble, are you familiar with any designs of railroad bridges with approximately 18 inches to 2 feet of ballast on them, say, four members or five members wide with no lateral prestressing and no lateral bolting; are you familiar with any in this country?

MR. RUBLE:

No.

QUESTION:

There are three bridges being designed in this country.

MR. RUBLE:

I know that, but they are not built yet.

QUESTION:

But, have they been examined by your association?

MR. RUBLE:

No.

QUESTION:

Their intentions were a hurry-up job. And, it is designed with the ballast, the rails and ballast on there with no lateral prestressing and proportional areas, proportional loading according to the areas of the member, taking their load on it, and I question the feasability of it. I see from your statement you questioned their wearing and possible differential deflections.

MR. BRYAN:

Yes.

MR. RUBLE:

Let me say one thing. I feel that they would work satisfactorily. In other words, each slab will be, say, two or three feet wide.

QUESTION:

Two and a half feet wide.

MR. RUBLE:

Something like that. I might mention that on steel beams, where we were studying the lateral distribution of load, with and without diaphragms, we could not find any difference. I doubt that in a very heavy section like we have here, if the diaphragm would be necessary.

QUESTION:

With ballast?

MR. RUBLE:

With ballast. Of course, most of them have ballast. Six inches is the minimum. I do not believe that you would have any difficulty with those slabs working properly.

QUESTION:

Ross, do you think the ballast has any effect?

MR. BRYAN:

I think your problem here is entirely different. It is not the same problem at all. You have a bridge with ties placed so as to distribute the load; you have a track going down on the bridge which confines the wheel loads to one position only. You design your bridge for that position of wheel loads. There, I would be inclined to say that you probably do not need a transverse tie.

MR. RUBLE:

If your load was shifted over a little bit and came over on another slab, that is a little different problem.

CHAIRMAN EDWARDS:

If you convert this over to a truck highway, it will be a different matter.

QUESTION:

That's right. I wanted to raise that question. Those are going to be built.

CHAIRMAN EDWARDS:

Would you like to make a comment?

MR. RUNDLETT:

You are referring to the sections abutted to each other and drag grouted?

QUESTION:

No, not drag grouted, just dry.

MR. RUNDLETT:

No comment, then, because I feel the drag grouting has proved very satisfactory from one section to the other.

CHAIRMAN EDWARDS:

Any other topics of discussion?

QUESTION:

I would like to ask Mr. Rundlett why you eliminated the T section in the standardization?

MR. RUNDLETT:

I think the fewer sections you have, the better off you are in this standardization. And, to me, the I-section is preferable to the T, and again, that is my personal preference. That is my only reason.

QUESTION:

Would that extend to lengths of 105 feet?

MR. RUNDLETT:

The T's rather than the I's.

QUESTION:

We were afraid that the I's would get lateral buckling. I wondered whether that is a real fear?

MR. RUNDLETT:

You have a top stem on your I, too; don't you?

QUESTION:

It is a question of widths.

MR. RUNDLETT:

Sure. The whole question is the proportioning of your widths.

CHAIRMAN EDWARDS:

I would like to ask a question to Mr. Smith.

In the application of prestressing to airport runways, we are going to get into more and more runways and thicker and thicker sections, more and more cement; so, pretty soon there will not be anything left over for prestressing use. But, the application that you mentioned is very interesting on runway work to cut down your thickness of slab and definitely improve the slab. Would you tell us if there is any definite progress being made to get more trial applications along this line or is it still in the strictly experimental and discussion group? Are we at the point where we can look to a greater use of this prestressing for runway work?

MR. SMITH:

Unfortunately, we have, as far as I know, no serious consideration of it in the United States. Those particular applications, the first one was in Orly Airport in Paris, and the subsequent application which was directly based upon the good results they received in Paris was out on the North African Airport, Maison Blanche, Algiers. As far as I know, in both cases, the slabs have stood up completely. There is no complaint on the structural adequacies of it, and the only possible consideration now would be the economy of it in the American market.

Now, if I could, I would like to ask Mr. Ruble if his society has made these beams specifically for railroad loading? The report I think you mentioned was going to be ready in about a year.

MR. RUBLE:

Yes.

MR. SMITH:

Just what can we, as potential designers and producers expect in that line, and would it actually mean an application of it, or is there a possibility that something like this might be pigeonholed?

MR. RUBLE:

Do you mean that the report would be pigeonholed?

MR. SMITH:

Well, the application of the report might be?

MR. RUBLE:

No. It will immediately receive wide distribution to all members of the American Railway Engineering Association. And, we will have reprints available. I think the principal advantages of the report is that it, more or less, graphically and physically demonstrates to the railroad bridge engineer that the construction of a prestressed slab or bridge slab is possible, and that it will function properly as designed. And, I think, they will have a lot of confidence in it if we can reduce the price and show whether it is economical.

MR. BRYAN:

I would like to ask Mr. Ruble a double-barreled question: I heard you say that the strength of this concrete in the slab was 8,000 or 9,000 psi; that the half-inch strand used was unusual in size for bonded strand; that the length of bond was developed in about six inches. I would like to comment that 8,000 or 9,000 psi concrete might be expected to do that, although that is a concrete that we normally would avoid using. I do not believe that volume construction would consist of 8,000 or 9,000 psi concrete.

The other question is: Several years ago, I believe a study was sponsored by your Railroad Association, and they were studying the effect of secondary stresses in riveted truss and bolted truss gusset plates, railroad gusset plates. In their study, which was very complete, they measured the impact of trains going across the bridge. You did not mention what your impact was, but they found, as I recall, the maximum impact they could measure was about 40 or 50 per cent, and it occurred at a speed of about 30 miles per hour. Would you care to comment on that?

MR. RUBLE:

Taking first the concrete strength in these slabs we had only specified the 5,000 psi concrete. However, the contractor decided to use a higher strength concrete so that he could get an earlier strength and remove the forms the next day. That was due just to production purposes there. And, it was left entirely up to him. Now, then, the use of one-half-inch strands, this slab was designed approximately three years ago, and at that time as far as I know, it was the first use of a half-inch strand. I think it would certainly be an advantage if we could go to a larger strand.

MR. BRYAN:

I do too.

MR. RUBLE:

As I said, those particular slabs, now, had 61 one-half-inch strands, and that is a lot of strands.

MR. BRYAN:

Would the half-inch strands have been satisfactory if your concrete had been 5,000 pounds?

MR. RUBLE:

Yes. I think it would have been, on account of the bond, you mean?

MR. BRYAN:

Yes.

MR. RUBLE:

We feel sure that it would have been. As I say, we developed those strands in the first six inches. Now, whether they stayed developed, we do not know. However, we do know that after about a year and a half, the camber increased slightly. Now, we do not know whether the plastic flow was greater than the loss in bond or exactly what happened; certainly, we felt better about it anyway. It was not a serious matter since it was just a slight increase.

Now, then, to get more information on the use of half-inch strands in railroad slabs, we expect to conduct some repeated load tests on beams similar to the ones that are tested here using various concrete strengths. We expect to take, 3,000, 5,000 and 8,000 psi concrete or three different concrete strengths and subject the beams to repeated loads of about 50 per cent greater than the design-load and possibly to two million cycles of load applications or something like that and keep track of our bond and make a detailed study of that.

Now, then, getting back to the impact in railroad bridges, of course, we have been working on that for 15 or 16 years now. And, I know less about it now than I think when I started. But, the particular thing that you are talking about was in tests of short-span bridges. And there with a battered rail joint at the center of the span, or say a flat spot on the wheel, your maximum effect will occur at a slow speed of 20 or 30 miles per hour for that particular case.

On a long highway span, it is a matter of synchronization, and the natural frequency of a long span is in the order of two or three cycles per second, maybe lower, depending upon the stiffness of the structure. And, there again your maximum impacts occur at a low speed.

Our new design equation has a maximum of 80 per cent impact factor. Now, that is for steel. For concrete, a function of dead load and the live load relation, we are securing data on impacts in concrete structures. We are not ready to say what they are yet.

CHAIRMAN EDWARDS:

Mr. Ruble, in my opinion the work that your organization has done has given a lot of encouragement to the use of larger strands, especially when we hear about the half-inch strand, the work that was done three years ago, and what you say now, since there was a great doubt about bond in prestressing. The Prestressed Concrete Institute cosponsored a project that was just completed at the University of Florida, conducted by Doctor Ozell who is here today, and some of the results in that test are very interesting on bond of seven-sixteenths inch strand where the indications are that if you do get loss at release, it makes practically no difference in ultimate

load or first cracking load in a normal beam. And, I think that what you have done has brought us a long way toward using the larger sizes. And, I hope we will be able to go further because there is a definite economy in the large sizes.

MR. DuBOIS:

I would like to answer your question on runways. About six months ago, a delegation from the Civil Aeronautics Administration and the Corps of Engineers went both to Paris and to North Africa and made a considerable investigation of what had been done. Information was brought back to Washington. And, I think, it is later than you think as far as the use of prestressed concrete in runways is concerned. The Port Authority in New York City is considering a design of a test runway, in Idlewild Field. I know that might give encouragement to your questions.

QUESTION:

I wanted to ask a question on bond. I heard a rumor, as a matter of fact, in the Navy Department in Washington, that some tests at Lehigh University were not showing up too well for long time bond of three-eighths inch strand. I wondered whether that was a correct report or whether anyone has any information on that?

CHAIRMAN EDWARDS:

Would anybody like to give us a report on that?

MR. RUNDLETT:

It's not three-eighths.

Carl Ekberg of Lehigh University is giving a report later in the convention on "The Characteristics of Prestressed Concrete under Repetitive Loading," and, I think, your question will be covered very well on Friday.

MR. RUBLE:

Tests made at the Portland Cement Association have shown that it takes around 12 inches to develop your prestressing value. However, those are in small specimens, and we feel that in the larger specimens, say, two inches of cover, it is not going to take as much, and you will not have the trouble in the real large specimens.

CHAIRMAN EDWARDS:

I think quite a bit of the field testing has indicated somewhere around eight inches for three-eighths inch in the larger specimens.

QUESTION:

Depending upon the concrete stress?

CHAIRMAN EDWARDS:

Depending upon the concrete and deformation. If you are going to let Poisson's ratio work on your steel, you are going to have deformation of your concrete.

QUESTION:

What do you mean by developing six inches or develop it in six inches? I do not understand your definition?

CHAIRMAN EDWARDS:

You will get complete bond between the prestressing steel and the concrete. At the end, if you do not have an anchorage, your steel has zero stress. And, as you go into the beam, your steel will have a greater and greater stress due to the bond that it has with the concrete.

QUESTION:

In other words, you have no bond at the end?

CHAIRMAN EDWARDS:

That's right. I believe Mr. Dorland may have something interesting on that. He has run quite a few experiments. I think he sent two beams over to Belgium to have them tested?

MR. DORLAND:

We actually use something that is rather unique. We use an end anchorage on a pretensioned product. It is an anchor that is stamped on the wire. This was developed around five or six years ago when we were using smooth rods and wanted to go to large sized wires. And, of course, you cannot depend upon the bond on those. We have run tests using this anchor on half inch and three-eighths inch and we use it on all of our three-eighths, although many times it is not necessary. We use it to insure the complete anchorage at the end for shear so we can depend upon the combined forces there when we develop shear.

A lot of times on short members where you have heavy shears, we feel it is very questionable whether you can depend upon your prestressing force to give you a combined tensile force. And, we use an anchor that is on the wire.

Now, we have run some pull-out tests, and found that for lower strength concrete it takes a lot longer distance to develop bond and it also depends upon your particular aggregate. In certain cases, I think it would also depend upon whether you had good placement of concrete around the strand. If you use too dry a mix, and you do not get good placement around your wire, then it could be rather dangerous. Using an end anchorage on pretensioned concrete may not be necessary in all cases but we use it mainly for the insuring of complete anchorage. Also, we use it to vary the magnitude of the stress. In other words, we anchor some of the wires so that we can vary the magnitude of the prestressing force, depending upon the designed force at that section. I think, that many of you know the advantages of this particular type of method.

We sent some beams clear over to Belgium to have them fatigue tested. This was on smooth wire, three-eighths inch diameter rod. I think the load was two and a half times the design-load. They ran them several million cycles. The wire failed in tension at the point of maximum moment, and there was no failure at the ends of the clips; but, these were short spans and we were interested in obtaining the anchorage at the end, so that you would have a good shear distribution. And, all of the beams failed actually in tension at the center and none of them failed at the end in shear or by failure of anchorage.

QUESTION:

I am very interested in the use of channels in small bridges, whether they are double-T channel or any other channels, for municipal and county bridges. First of all, I would like to ask if your bridge had a topping placed in the field, what the trend is for that, and also what success you have had?

MR. DORLAND:

We did top ours with asphalt. We were a sub-contractor, and we only erected the bridge. It was not necessary to top it, but this was an alternate bid, and all we did was supply the alternate material to carry the roadway. And, according to the specifications, the wood deck bridge had to have a topping of asphalt. And, since it was in the specifications, they did top the concrete deck with asphalt. However, after the joints were grouted, they were uniform enough so that it did not need to be topped. Actually, about two months later it was even rougher than it would have been had they left it alone.

The asphalt became much rougher than the original surface. I do not think topping is necessary if you get a good uniform surface without too much camber.

QUESTION:

What was the thickness of your deck on those?

MR. DORLAND:

They were five inches. That was required in order to have an inch and a half coverage over the steel, since it was over water. The depth from the center of the steel, I think, was three and a half or four inches.

CHAIRMAN EDWARDS:

The point that Bill brought out, I think, is a very good point on County bridges and secondary bridges. There are a lot of good ones going in of various types and some of experimental nature, and I think eventually we will develop a secondary bridge that will be entirely different than our main bridge structures.

The construction will be very fast. You will set them in and move out, and will be finished in one day and all through with the bridge, and that bridge will hold up. In fact, a lot of them going in today are excellent bridges, better than anything that has gone in before on secondary roadwork for the County roads.

MR. RUNDLETT:

Perhaps we do not work as closely up north as you do down here, but I think we would always figure that if you want a good riding surface, you better top it.

MR. BRYAN:

I do not believe in sending a boy in to do a man's job. I do not believe that you can use a roof channel or double-T or floor channel or floor double-T as a bridge section. I think a bridge channel-section is probably one of the most economical. I would be safe in saying that the most economical bridge section spans up to about 50, probably 45 feet. I think it probably should be around four feet wide, as Mr. Rundlett suggested. I think it probably should be three legged, and I think that it probably will have to be topped. I do not believe we can make it so that the camber will be uniform enough to do away with the topping. I think they should be grouted and should be transversely prestressed.

CHAIRMAN EDWARDS:

By topping, did you refer to a concrete topping?

MR. BRYAN:

Asphalt.

CHAIRMAN EDWARDS:

I think there is quite a bit of trend toward asphalt topping. There is a program being started in South Carolina on secondary roads. I think it is going to be interesting to watch that. It is made with a channel section, and it is bolted laterally, I do not know whether you can give it the name of lateral post-tensioning, but it has rods running through laterally which can be taken up. The joints have a V section, and I think that they are going to use an asphalt topping.

● memo to advertising managers

ADVERTISE your materials, equipment or services in the PCI Journal, the only publication covering the B-I-G prestressed concrete industry. The JOURNAL is the official technical publication of the Prestressed Concrete Institute, a national organization dedicated to the advancement of this booming new industry. It delivers concentrated buying power . . . circulation where it counts.
Write for advertising rates, mechanical specifications.

PUBLICATION OFFICE
3132 N. E. Ninth Street
Fort Lauderdale, Florida

Second Annual Prestressed Concrete Institute Convention

PANEL DISCUSSION

Production and Inspection

CHAIRMAN

P. J. Verna, Jr., Concrete Materials, Inc., Charlotte, N. C.

PANEL MEMBERS

Werner Rosch, Lewis Manufacturing Company, Miami, Fla.

E. K. Rice, Western Concrete Structures Co., Los Angeles, Cal.

Harold A. Price, Basalt Rock Co., Napa, Cal.

J. C. Rundlett, Bridge Engineer, Dept. Public Works, Boston, Mass.

Donald A. Sawyer, Civil Engineering Dept., University of Florida, Gainesville, Fla.

MR. VERNA:

Our first speaker will be D. A. Sawyer, Instructor of Civil Engineering from the University of Florida. His topic will be "The Role of the Engineering Inspector."

MR. SAWYER:

There have been a great many jokes, I think, circulated about the engineer-and-contractor relationship and, unfortunately, there has been a great deal of truth spoken in jest and I think we all realize that there is, to a certain extent, a certain amount of divided interest between the engineering inspector and the contractor.

In general, there are two types of inspection: That which the producer does himself to insure a good quality product and the other done by the engineer to assure that his client is getting a job according to the specifications. In speaking of inspectors, I also mean the engineering staff in general, and all the way up to the designer in some cases.

The engineer-inspector on a prestressed job, to a great extent, is no different than the inspector on any other job who is working with concrete. He should know his concrete from the aggregate to the last job of curing. He should know about his aggregates. That is, how the gradation requirements will effect his mix and all the things about mixing and placing the concrete. He should know curing. He should know the effects of admixtures and should be aware of the fact that admixtures can be dangerous. I recall hearing about an inspector who was not well trained and did not know exactly what he was doing on a job, which, I feel, is the case too often. He was using a certain admixture and got behind in his dosage. He thought he had better catch up so he over-dosed a few batches in order to catch up and caused a terrible financial loss to the contractor because a whole slab had to be torn out.

In that respect, I think inspectors should be well trained in all phases of concrete mixing and quality control. I think the inspectors should know the importance of making the units, with all the care it takes to insure that the aggregates are right and his mixing is right and his placing is right. Then, if he ends up making his cylinders sloppy, the whole thing is for nothing. He may say that the concrete is bad because he himself was at fault in making his cylinders. He should also make his cylinders properly and cure them properly. In the early days the steel used in prestressing was a strange material to the contractor and to the inspector. Now it is a more common material. Nevertheless, the inspector should know the properties of the steel he is using. He should know how close he is to certain critical points on the stress-strain diagram. This is expecting a lot of the inspector, but I think he should know it. In inspecting a prestressing operation, I think, he should know all the details. A great deal of prestress work now is done by pretensioning. However, I think the general engineering inspector should expect to run across any one of the phases of pretensioning or post-tensioning. He should know all the factors involved where he can go astray.

In the prestressing operation itself, I think he should know the importance of cable positioning. I do not believe that it is quite as critical as was thought in the early days when prestressing was first started in this country. At that time, we were worried that it was going to be a tremendous job for the inspector. It is not quite as critical as we thought.

I think that in I-sections, the inspector should particularly be aware that slight horizontal displacements of his cable may result in quite a horizontal camber and in twisted sections. A very slight amount is an alarming proportion of the allowable distance.

I think the hardest problem an engineering inspector on a prestressed job has is knowing just what his tolerances are. Mr. Rundlett will speak on tolerances, but I would like to mention just a few hints that might be well for the engineering inspector to know and these things should be passed on to him by his superior. However, he should also be given some leeway.

I remember a job on which I was a member of the inspection staff — a post-tensioned job. We were not handed any tolerances at all. We descended upon the contractor with micrometers and promptly started micrometering all his bars. We had elaborate tables set up indicating that, for example, a bar 0.995 inch in diameter had to elongate 2.38 inches at the tensioning load. That was it. Of course, we did not get the exact elongation for the particular load and I can recall some correspondence back and forth with the main office about what in the world was the matter. I think the designer particularly should realize that he can not specify an exact elongation.

The tolerances that an inspector might work with in pretensioning are much different from those in post-tensioning. In pretensioning, the beds are very long. The total elongation is about 20 inches or so, depending upon the length of the bed. A quarter of an inch difference in the elongation is practically nothing. The cable is free and has no friction to resist it. So you should expect a fairly good check. There is some danger in that it may not check exactly because the modulus of elasticity varies. I think, on the average, you may expect certain variations in post-tensioning. However, it is very hard to say what should be your tolerance. Most post-tensioned girders are relatively short. The total elongation might be two and a half inches or less, hence, a quarter of an inch is quite a large proportion of the total. You know you have friction if it is a curved cable or bar. Slight kinks in the cable will cause points of high friction.

Also, in post-tensioning, the concrete is shortened during the stressing operation. The amount of shortening depends a great deal upon the modulus of elasticity at the time you prestress. You can not expect an exact check. I think the inspector should have a working tolerance on this. He should know how much he can expect. In the new A.C.I. recommended practice for prestressed concrete, there is a nice neat little formula for comparing the actual elongation with the theoretical in order to compute the friction. This is very interesting, but so what? If the inspector on the job gets a discrepancy, he can compute how much friction was present. The point is, is that too much? The inspector should be aware of the overall picture of the job. He should start from the beginning and carry the job all the way through.

He should not let a contractor install his cables or his bars in such a way that they will get displaced badly and then try to reject the product on the basis of too much camber or not enough elongation, when he himself was lax in letting the contractor go ahead when he was not ready.

Finally, I think the inspector has certain responsibilities which he might not realize. Of course, he has responsibilities to his employer and to the client to see that the job is done according to the specifications. Also, I think, he has a responsibility to the contractor. I think the contractor should expect and get a certain amount of understanding from the inspector. The inspector should realize that the contractor has a certain selling price for his material and the difference between his production price and the selling price is his profit. Anything that the inspector does unnecessarily to shorten that profit should not be done. Any arbitrary decision he makes which costs the contractor money is unfair to the contractor. And from that standpoint, it is again a matter of tolerances. He should know what his tolerances are, he should tell the contractor what his tolerances are, and then he should give the contractor free rein within those tolerances.

Now, finally, you might say that, perhaps, there is no such inspector who knows all about prestressing, knows all about concrete work who appreciates the contractor's point of view and tries to go along with him. However, I think that a great deal can be done by the engineer if he knows just what can be expected and pass this on to the inspector. The engineer should be more careful in choosing the inspector. I have seen inspectors that, as far as I was concerned, would better never have been hired.

MR. VERNA:

The next speaker is Mr. H. A. Price from the Basalt Rock Company of Napa, California. His topic will be: "Facilities and Equipment."

MR. PRICE:

Most prestressed concrete products plants have been started on a small scale and grown as necessity dictated without the advantages of overall planning.

Among the deficiencies most often observed are:

Property

Engineers talent

Testing and research facilities

Aggregate and cement storage facilities

Batching and mixing equipment
Mix-transporting and placing equipment
Jacking equipment
Universal tensioning beds
Vibrating equipment
Curing facilities
Lifting equipment (cranes, lift trucks)
Transportation equipment.

Time will not permit a complete discussion of each of the items mentioned, but to stimulate discussion for the question and answer period to follow, I will try to give you some of my thoughts concerning some of the elements I consider essential to an efficient operation of the plant if it is to grow and prosper.

PROPERTY:

Just what the optimum acreage should be depends to a great extent on the products to be manufactured as well as the marketing area to be served. It is well to remember in this day of tremendous military and civilian construction projects, that it is possible to sell materials for a job of such size that new facilities requiring as much as five or ten acres may be required for production and storage. Our own present facilities are:

3.4 acres, plant and warehouse
.4 acre, prestressing beds
5.0 acres, storage
2.0 acres, future prestressing beds
4.0 acres, future storage

14.8 acres, total.

This is by no means the ultimate that we will require.

ENGINEERING TALENT:

This cannot be classed as a facility, but it is a requisite to this business. Talent is extremely hard to come by, a fact which cannot be overlooked and must not be underestimated. We have three civil engineers, five draftsmen and two estimators. In addition, we have two civil engineers supervising erection and our Production Manager who is a mechanical engineer.

RESEARCH AND TESTING LABORATORY:

An adequately staffed and equipped testing laboratory is certainly a necessity. I cannot conceive how we could possibly operate without our lab. Its principal functions related to everyday production are:

1. Mix design
2. Control of mix at batching plant (compensating for free moisture in aggregate and checking yield)
3. Testing of aggregates
4. Determination of curing cycle
5. Making of test cylinders
6. Testing of cylinders to determine compressive strength so the plant will know when members can be tensioned.
7. Calibration of jacks and pressure gages
8. Certification of product for compliance with the specifications
9. Quality control.

Some of the equipment used in our laboratory are:
300,000 pound Baldwin testing machine
70°F, humidity curing room
70°F, humidity control room
Drying oven
Autoclave
Gilson screen
Ro-tap screen
Whittimore strain gages
Anderson Strainmeter
Carlson gage
Paddle-type and pan-type concrete mixers.

AGGREGATE AND CEMENT STORAGE FACILITIES:

It is possible to make prestressed concrete by obtaining concrete from a nearby ready-mix plant. The factors of economy and necessity for quality control will, eventually, demand adequate aggregate and cement storage, batching and mixing facilities at the yard. Batching equipment must be accurate. Much can be said about mixing. I believe we were the first in this country to use the horizontal-pan English-type mixer. This machine produces concrete with about 15 per cent more compressive strength than the same concrete mixed in a conventional drum type mixer. Naturally, savings in cement are possible. It also will mix and discharge no-slump concrete which we feel is the ultimate aim for all prestressed work.

JACKING EQUIPMENT:

There is not too much to say about jacking equipment except that it seems we are always needing new equipment from 30-ton 3″ stroke up to 300-ton 48″ stroke. Improvements

are being made which will reduce operating costs. The new Freyssinet "H" type automatic wedging jack is far advanced over any previous model for similar work.

UNIVERSAL TENSIONING BEDS:

Tensioning beds are too often designed around one or two particular products such as piles or beams and do not have the flexibility to be readily adaptable to new applications. Frequently, they are too long to produce a small job without excessive form cost. We have one prestressing bed that might be of particular interest. It is 27 feet wide by 180 feet long with an intermediate removable abutment at 80 feet from the stressing end. It also has a "let-down" device which permits us to shorten the bed in 4-foot increments up to 40 feet from stressing end or from either of the semi-fixed dead-end abutments. This bed is truly "Universal."

VIBRATING EQUIPMENT:

Vibrating equipment is an essential part of every prestressed concrete products plant. Pneumatic and electric types for internal and external application are both satisfactory. The pneumatic type generally has the advantage of higher frequencies which is sometimes desirable. It has been our experience that when large numbers are used an ample source of air can be something of a problem. We have also found maintenance cost to be higher for the pneumatic type. Our investment in vibrators exceeds $10,000.00.

CURING FACILITIES:

The heart of our curing facilities is always a steam generator or boiler. Since the actual requirements are almost impossible to compute, most producers have found that the boiler originally selected was too small. The flash type boiler does not usually produce sufficient steam. We have found that peak loads for our present operation require 200 horsepower. We have a horizontal drum water-tube marine-type boiler. Additional capacity will soon be required.

LIFTING EQUIPMENT:

Lifting equipment is almost always too small for the next job. Inside our plant we operate two 25-ton capacity overhead cab-operated cranes which seem adequate. For operation outside the plant, in storage areas and particularly on the pretensioning beds, we have found that two 10-ton lift trucks are more versatile than a single 20-ton mobile pneumatic-tired crane.

TRANSPORTATION EQUIPMENT:

To deliver the large prestressed concrete members, which we are now making in increasing numbers, special transporting equipment is needed. At present, we have equipment capable of transporting 40-ton girders 80 feet long over California highways. For the present, we have reached the limit but as we progress, new equipment and new methods will have to be developed.

We have an arbitrary ruling by the Division of Highways which will not let us have a prime mover over 110 feet in overall length and it is pretty much of a problem to move a girder over 80 feet long with a tractor.

If I were asked what expenditure were necessary for facilities and equipment to produce prestressed concrete, I would find it difficult to reply. I suppose we can all agree that regardless of our progress in this industry, it is only in its infancy. I believe it will grow in much the same way as did the steel industry, and eventually we will have to make similar investments. They will not be measured in thousands of dollars or hundreds of thousands, but in millions.

And the best advice I can give you is this. Be sure you have plenty of property available for expansion that is strategically located for rail and water shipments as well as for highway transportation.

MR. VERNA:

Mr. J. C. Rundlett of the Department of Public Works, Boston, Massachusetts, is going to speak on the topic of "Tolerances – Dimensional and Strength." I might add, gentlemen, that that seems like an inconsequential title for a topic, but it has been our experience – it is the feeling of my company that tolerances probably will be one of the major determining factors in furthering progress in prestressed concrete. Because of all the work constructed today, I expect there has been more discussion about tolerances than anything else.

MR. RUNDLETT:

The subject assigned to me, "Allowable Tolerances in Dimensions and Strength of Prestressed Units as They Affect the Designer, Owner and Manufacturer," is one that I would much prefer were assigned to someone else.

The impractical designer insists, because we are dealing with stresses higher than those used in conventional work heretofore, that everything must be manufactured mathematically perfect. One has only to observe a casting and stressing operation to know that such a requirement is impossible, and one with common sense will accept the fact that such units cannot and need not be constructed to a watchmaker's precision.

The "What the Hell Contractor" will say: "Just tell me what you want and leave me alone, I will build you a beam or a thousand beams. You engineers are too fussy and you don't know what to do or where to stop. Get off my neck." One has only to visit his casting bed, or better still his dump pile, to see a few distorted, cracked, or broken beams to know that his is not the right approach.

The owner declares: "Give me beams that look well and will carry the load and still will not cost me anything and I'll be happy. You don't need all that steel, leave it out if it's going to add to the cost." This man, too, is trying to land on the wrong runway. These examples, you and I know, are the extremists which you meet wherever you go, and they are not repre-

sentative of any of the groups interested in prestressing.

Institutes, such as yours, are founded to bring those interested together to exchange ideas and to formulate policies that are correct and reasonable and acceptable to all, or at least to the great majority.

The introduction to the Preliminary Report of ACI-ASCE Joint Committee 323 on Recommended Practice for Prestressed Concrete reads as follows: "Construction of prestressed concrete members demands a greater degree of care and precision than that exercised in conventionally reinforced concrete if successful results are to be obtained. This is evidenced from the fact that much thinner sections having higher concrete strength must be employed to obtain the greatest economy. It is, therefore, essential that the concrete in each and every unit of a prestressed structure be a homogeneous mass, sound and enduring in quality."

To all this I agree, and my remarks could close at this point by adding the following: With reasonable care, units manufactured in a plant especially designed for prestressed work with well-constructed beds, well-designed and built forms, by average conscientious plant workmen, will automatically be better controlled and better constructed than units of a conventional type made in the field where working conditions are far from ideal and where operations are many times makeshift.

Most manufacturers are now aware of the need for carefulness. Many have had unhappy experiences and have had serious injuries due to their lack of respect for strands, bars, or cables under high tension. Many have had rejections of members and some failures because of improper alignment and stability of forms and because improper care was taken in the lateral positioning of the stressing units.

Prestressing is no occupation for the man who does not recognize that the only way to maintain a successful and profitable business is to do the work right in the first place so that he will have no costly rejects or failures or accidents.

If the concrete is manufactured according to the specification requirements, if the forms are straight and adequately braced and supported, if the prestressing units are placed where called for on the plans and stressed according to directions, and if the concrete is placed and cured according to accepted practices, there will be no question of tolerances or acceptance.

From my observations, the cause of rejection of a beam is usually instantly apparent and no complicated measuring devices are required to confirm this judgment.

Comparison of like units, I believe, can be the only measure of tolerances. The camber induced and the shortening of the beams will not be exactly as calculated but will follow a general pattern. Any black-sheep beam which does not follow the pattern is obviously defective for one reason or another and should be discarded.

I assume that I am expected to recommend some definite mathematical tolerances on dimensions and strengths, but I regret that I cannot make such a profound announcement. (A yardstick at the present time could be that if inaccuracies are such that the allowable stresses are not exceeded, then the beam is satisfactory.)

To me there is no better contract than that which calls for the beams to be constructed according to the plans and specifications, to a true line and section, and according to the best acceptable practice. Neither the engineer nor the contractor familiar with prestressing are unaware of what that means, and under it there is no excuse for delivery to the owner of prestressed concrete members of inferior quality.

MR. VERNA:

Our next speaker is the gentleman whom we have asked this morning to fill in for Mr. Bruce. Mr. Bruce, from Canada, cancelled out at the last minute due to some unforeseen commitments that came up. So, Werner Rosch of the Lewis Manufacturing Company of Miami, Florida, has graciously taken the topic on "Curing." I want to apologize for him should he not be able to keep you entertained and give you some good information on such short notice (he was given this job an hour ago).

MR. ROSCH:

The curing methods used in concrete work are, in general: water curing, that is, moist-sand or burlap, integral admixtures sprayed on, membrane curing, and heat or steam curing. Water or wet burlap curing is applicable mostly for general construction work. But in the prefabricating business, where we have to turn over our benches faster, we have to search for a faster curing method. Some yards have been using integral admixtures, such as calcium chloride and so on, but there has been evidence that calcium chloride has caused corrosion, at least we can term it this way: Calcium chloride in combination with steam has shown indication of stress corrosion. The sprayed-on membrane curing is used as an initial protection on large-surfaced members. I use it myself because if a pour is made, say early in the afternoon, the sun will affect the surface so much that we have to protect it. Our main curing method is by warm water or steam curing. This is important because we have very expensive forms. I would like to mention that, for instance, a double-T form costs us between $10,000.00 and $20,000.00, depending on its length and how it is manufactured. These forms have to have a maximum turnover for economical reasons. The best method of curing is to cover the member with tarpaulins and blow live steam from underneath. Many yards cannot afford that. We work in the open and the storm and wind in Florida during the summer season will tear off the tarpaulins or the covers. So, many yards are using heat from underneath the forms and have a sprinkler system working on top. I have some information about curing which will be of interest to you.

The Prestressed Concrete Institute and the Engineering and Industrial Experiment Station of the University of Florida cosponsored a research program on static and fatigue strength of prestressed concrete members; and one of the tasks of this study was to find out the difference between the wet burlap curing method, and the steam curing method. The quality of a pretensioned member is mainly affected by a combination of stress in the strands, the compressive strength, bond, and the modulus of elasticity of the concrete. How these different values were affected during the tests is reported in one of the papers which is now available.

The modulus of elasticity of steam cured test specimens has been better than those of wet burlap curing, which means that steam curing saves you time and allows you to strip the member earlier out of the form. It not only allows more production, but it gives us a better product.

Now, the main thing is this: When you release the strands from the anchorage and apply the prestress to the member, the tests show that the steam cured materials behave slightly better than moist cured members.

MR. VERNA:

Our next speaker will be Mr. E. K. Rice from the Western Concrete Structures Company of Los Angeles, California. His topic shall be: "Safety in Production, Transportation and Construction."

MR. RICE:

Very often, safety is a topic that some people regard as an old woman's harangue all the time, watch the signs, watch your fingers; that type of thing. However, I think we can look upon safety as something that actually makes dollars for us. I will speak primarily about safety to men and also safety to the units themselves.

Safety to the men, of course, is very important to us as our men are hard to come by. Also, our compensation insurance carriers say, "Thou shalt be careful or thou shalt be priced out of the market." As far as safety to units goes, I think we have a problem in cast-members. It is a little unique in that very often, if during erection we have the unfortunate experience of breaking a member, the whole project can be held up for several days and, in some cases, several weeks. This is not true with many other forms of construction materials. So, it behooves us to take extra care with our members once they are cast because they are often difficult to replace, particularly if you have torn the forms out.

I do not think we need to go into a discussion of the so-called usual hazards of concrete construction work. These are the smashed finger and the nail-through-feet type of thing. I think, though, we want to discuss the rather unique hazards in the precasting-prestressing business.

The first thing we cannot overlook is that we are dealing with very large forces, and after we transfer these forces to the members, they are active forces. They are not passive forces as we have in other members. We must learn to respect these forces.

We might discuss wire handling and stressing for a moment. By wire handling, of course, we mean putting the wire on the bed. Stretching, of course, is stretching the wire, whether it be in the plant or a post-tension operation. All of us, I think, realize that we are dealing with very large forces and at times, we lose respect for those forces. That is when we are in trouble and we have got to continually remember that this force can get away from us. What we must do is to look at our abutments, our pull rods and all of our pulling gear to see what would happen if some of this gear should break.

We have been acquainted with a couple of breakages of gear of that type. One break was of an I-beam, a fairly deep section that was used to spread a bar to pull wire with. Someone became careless and went out and chopped off a piece of I-beam and stuck it into the ground without too much thought and neglected to put some stiffeners in the I-beam. When the load was applied, the I-beam snapped like an accordion and the pull rods were down on the bed and tore quite a bit of the form work. Fortunately, no one was injured, but it was a good lesson on what tremendous forces we are dealing with.

The next thing is this matter of pull rods. We also have witnessed a failure of a pull rod and it can produce pretty bad results, as far as tearing up form work goes. It will go down the bed like a slingshot and it will tear up quite a bit of the material. I think that it is worthwhile to take the trouble to periodically proofload your pull rods. In the case of heat-treated rods, sometimes, for various reasons, the quality control might not be quite what it should be. Also, some types of heat-treated rods are a bit sensitive to the unusual beating-around that the materials get when you have construction men working with them. I think it would behoove most of us to, perhaps, proof-load our pull rods and pulling mechanism once a month or so.

The next type of trouble we seem to get into is the matter of wire breakage. I think that it has been reduced a tremendous amount from the early days, mainly due to very high quality control on the part of the people who manufacture the wire. The second thing is the trend from solid wire, solid bars, to cable or strands. The danger in using the solid bar or solid piece of wire, is that if it does break, we have the very sudden failure and the wire is free to spring down in the bed. In the case of a strand, you only break one wire and it is usually held by the rest of the wires. You only lose a little bit of prestressing force but you do not tear up the forms.

We have used a considerable number of rods, mainly for tying precast members together. The rods cause the same difficulty in that if a post-tension rod breaks in the field, very often the jack will go off flying into space and again we have the very dangerous missile.

The next thing we might talk about is properly designed pickup points. We have done quite a bit of precast work down through the years in Southern California and people get a little bit careless about pickup points once in a while. They like to stick in pieces of fairly hard reinforcing steel in the shape of a hairpin and call that a pickup point. Every once in a while the reinforcing bar gets broken off. If they are lucky, they do the breaking before the member is picked up. Occasionally, however, they destroy a member. So, we must be very careful to have carefully designed pickup points. We must remember, of course, that things are not always perfect in the field. Occasions do arise when unusual stresses are placed on the member during handling. We have to have safety at the pickup points. We rarely get a direct pull on a pickup point.

Next, is the matter of stockpiling members. I think this is mainly a matter of common sense. We often have green members. They must be properly cushioned or we will do some damage to them.

As far as transportation goes, there is not too much that can be said about unusual safety requirements. We do not want to overlook the fact that we have got to properly support the member during transportation. The average person does not realize that we have to support the precast-prestressed member in somewhat the same manner that it would be supported in its final condition. If you are transporting a member that is going over three supports, you have got to support it on the truck bed in a similar fashion or you will get into trouble. I think all of you are familiar with this problem.

As far as safety in construction is concerned. I think, that all we can say is that those of us in the prestressing business have got to take time out and go through a little educational program with the men in the field. We have got to brief them on what the unusual hazards are that come up in prestressed construction.

This was brought home to me very suddenly last week. We were out erecting 122-foot bridge girders and we have to have good crews on the job. The crane operators looked as if they knew what they were doing, so, very little briefing was done about the type of members they were handling. They mishandled the member. It went over on the side and destroyed itself. It was a rather painful experience, but it is one that is certainly in my mind and I think we will never go out again and erect members without knowing what the particular problems are with regard to prestressing. This is particularly so in the matter of having to maintain the dead-load on a member to keep it from being broken.

Of course, this is very elementary, but you cannot expect the average crane crew to recognize the hazards of handling. It is our responsibility to brief them and let them know what these hazards are.

MR. VERNA:

In concluding the various topics, I will go into the production and labor costs of standard and non-standard units and also the material and product inspection. This will more or less cover the field of actual production. I will give you the experience of my own company since it is the one with which I am most familiar.

The prestressing companies in this country are not all of a similar nature. It has been our experience that we are geared to a particular type of prestressing – mainly warehouses and commercial buildings. We specialize primarily in double-T roof members, precast columns, precast-prestressed inverted T-girders to form a package of columns, girders and roof-decking material.

We started out making various different materials, flat slabs and joists and just about any kind of member that anybody wanted and asked us to make, we would try to make them. After about two years of operation, we suddenly realized that we had lost too much money. We were almost out of business. So, at that point we decided to concentrate upon a basic structure involving double-T, columns and girders sold as a composite unit because in our particular area we have been able to gear ourselves to that. The need in our area has been for one story multiple bay construction. Other areas may not have that need; they may have other needs.

However, even within that area of influence of our production facilities, there are architects and engineers and owners and builders who want channel slabs, flat slabs of short span capacities and various other sundry precast items. We have attempted to serve them, but found that our personnel lose time if we deviate from the standards of the columns, girders and double-T's which we produce daily. It has gotten to the point now where we are almost turning down or refusing anything other than our standards. In the future, we hope to be able to develop personnel who will be versatile enough to handle the unique and separate and different items that the consulting professions desire and really need; but, I think, that it is a mistake for a prestressing company when it is getting organized – and we are still organizing even though we have been in existence for two and a half or three years. We are still organizing and constantly changing. The problems of actually directing our men are quite complex. Experience has shown that when we deviate from the standard unit, our costs and labor almost double. This is so much so that our projects of the non-standard production units have almost caused us to shriek in horror at our stockholders' meetings. So, from that standpoint, we have standardized on three or four items.

Now, even though our men make the same unit, day in and day out, we still cannot make those right. We have been forced, of course, to develop personnel who can independently inspect all items in our stockpile area prior to shipment. In fact, even with the rush of business

as it is, if we do not do that, we regret it every time because squareness, true length and true width is a problem. Putting a multiple bay structure together is also another problem. If you do not really rigidly pre-inspect your own products, you are just more or less selling yourself down the river. You have to do it. It is just a demand requirement.

Now, I want to speak on one other phase of production. It has not been covered yet but here is some food for thought for all of you. Some of you are wondering whether to go into the prestressed business. Some of you are here, trying to further your own prestressing production facilities.

There are two aspects of production. You can, if you have enough money, equip yourself with sufficient forms so that your crew can be kept busy five, six or seven days a week, depending upon the needs in your area. It is a must that your full-time crew be kept busy every day. If you have to wait an extra day and do not have anything for that crew to do, your crew is producing only every other day. You cannot economically compete with steel or other precast people. If you have enough money you can have enough forms so your crew can be kept busy. In my firm we do not have enough money. We have to have our forms in use every day and pour every day in the same set of forms. We have had to gear ourselves to what we call a 24-hour-cycle. To do that, you have to have adequate curing and it takes expert control. We operate under a 4000 psi concrete stress at release. We have to check daily the production qualities of our concrete.

We have a steam operation that costs us approximately two per cent of our total sale price. That includes the fuel for the burners, the depreciation of the burners, the pipes for the distribution system and repairs on our boilers — which can be quite large. This is particularly true in our case due to the fact that we bought old boilers made 10 years ago and in patching or repairing the canvas which invariably burns while cutting strands. You just cannot throw them away. You have to repair them.

I want to bring out this production thought from two standpoints. If there are two sets of forms your crew can be kept busy every day, turn over every other day, seven days a week, 365 days a year.

QUESTION:

I would like to ask my first question of Mr. Rice. A year ago there was a discussion, pro and con, on using overhead cranes or using crawler-type cranes in a prestress operation. I should like to get your views on which you think is the most necessary.

MR. RICE:

I can answer that only by giving you our own experience. At the present time, our building in which the overhead cranes operate is 1,056 feet long. It is a 75-foot clear-span building. There are no prestressing beds within the building. We do a great deal of post-tensioning in our area. We post-tension them because we have facilities to handle them and we could not handle them economically if they were pre-tensioned because there are no facilities in that end of the plant. We do plan to extend our crane-ways and eventually add additional cranes to service the prestressing area. However, we have a problem of getting material out from under the crane which means some method must be found of loading a long member transversely to the 75-foot span.

We like to use two lift trucks which give us a total lifting capacity of 20 tons and almost unrestricted movement. There are a number of tricks you learn and means that you attempt in order to get things out of the cramped spaces. We developed a couple of lifting cranes that have a capacity of 25 tons each, so that we can unload members in the yard and then pick them up and load them on the truck without having to have a crane. We do not like cranes because cranes are a two-man operation, normally. We operate fully under a union contract and a crane operator, I believe, makes just under $3.00. His other operator earns about $2.65, or in that neighborhood. It makes a rather expensive operation and they do not move too fast.

To answer your question directly, I think we want overhead cranes for as far as we can use them and we will use only mobile cranes when we have to get heavy units out of our prestressing area.

MR. VERNA:

It has been our experience that an overhead crane is the fastest thing we can put in — an overhead crane on a rail system — and our area is not paved. We were fortunate to be able to pick up a pretty good capacity crawler. It was written off by a contractor at a pretty good price and it has ample capacity for the material in our area. So, we use that very satisfactorily and very speedily as a yard crane. If we had sufficient money, we would go and get a $60,000 or $70,000 unit with large rubber tires on it and about a 35-ton capacity on about a 20 or 25-foot radius. It would be self-propelled for practically a one-purpose use of the yard. We think that is the ideal situation, but we have to grow to that point.

I think, you have to study the problem of material handling from the hardened state to the stockpile area and shipping for erection.

We started out wrong in our operation. We tried to do just everything, all in one small area. We had to abandon and cover up old things and we have evolved what we think is a pretty good idea. Within the past three weeks we have completed a railroad track that runs right through our overhead crane-way. We have the rails for the rail siding right through the double-T bed. It is parallel to our piling bed, of which we had one. Our overhead crane was too busy trying to move down with finished products to load on to the trucks. We made it so

that our trucks can drive through in the same place. We built little dollies. We load on to these dollies so all we have is no more than a 25-foot movement of our crane trolley and we can push that. It is practically dead flat. We shall push those dollies down as we go so our time of stripping will be cut down tremendously.

QUESTION:

Wouldn't you say that an overhead crane-way is necessary?

MR. VERNA:

When you are not producing more than a thousand square feet a day you can get by, I think, with a crawler crane; but, if you get up to 5,000 square feet a day and you are handling not six or ten pieces of material, but make upwards of 20 to 50 pieces day in and day out, you want speed. You cannot play around with a crawler crane. You cannot play around with a fork lift. You have to get in there and move the stuff. It is my opinion that when you get into volume production, you need an overhead crane.

QUESTION:

I would like to ask Mr. Rice a question. Are your pretensioning beds in the open now, directly in line with that 75-foot span building, so that all you need to do is extend the crane-way over the beds?

MR. RICE:

That is right. We have one problem. There are two cranes on the track and as we go along, probably we will add a third. Getting one crane to pass the other, which is sometimes desirable, is going to be quite a trick.

QUESTION:

I would like to ask Mr. Verna, and possibly some other members, on their experience with temperature for steam curing.

MR. VERNA:

A very good question. I am glad you asked that because we have been steam curing for about a year now, and it has been our observation that the optimum delay varies before the application of steam, with the season and with the temperature of our mix. Now, in the wintertime our temperatures in Charlotte, North Carolina, from November until March, will be from about 30 to 45°F. The concrete does not set too quickly then and during those months we delay three hours from the time of mixing. Now, that is mixing, not pouring. Our concrete ready-mix plant is within 500 feet of our prestressed plant. So, there is no delay between the mixing and the placing of concrete in our operation. We delay three hours before application of steam. Due to the fact that we are in the open and we have only canvas covers over our concrete members, our rate of temperature rise is not too rapid. I might add that we cover our material immediately upon pouring, so that the canvas is over it. The steam pipes are in position and we just wait for time. The heat of hydration keeps it from being cool. It gradually builds up its own heat. At the end of three hours during the winter months, we apply our steam. Due to the fact that we have so many steam lines out, we do not get too rapid a temperature rise. We checked it and it takes us three to four hours to get the temperature up to about 120 to 140 degrees. Approximately 15 hours later we stop the steam, release the tension, strip out and set up again. So, approximately every 24 hours a bed is poured.

In the summertime, we get into hot weather. Yesterday, it was 96 up there. That concrete sets pretty fast, because we use Type III cement, and we use steam about an hour and a half after it is mixed. Also, we find that due to the fact we are in the open, the sun gets pretty hot. By covering it with tarps, and shading our concrete in the summertime, we tend to decrease our surface moisture checking due to the rapid set. So, we vary time delays and have found that when we apply our steam immediately upon pouring, we do not get nearly as high a strength as we do when we wait three hours and then apply our steam. Concrete that has 15 hours steam with three hours delay will be 20 per cent higher than concrete with 18 hours steam.

QUESTION:

We are in the concrete business and have found that heat has been better than steam. We are to begin a production of double-T's next week and we are planning to use this hot-oil system. I wonder if anyone in this room has any experience in this line. We have a double-T bed.

MR. VERNA:

You are going to pipe it through underneath?

QUESTION:

Pipe it through with the pipes on each side.

MR. VERNA:

Well, it is my experience that the piped hot water or hot oil or steam in pipes without moisture control on the surface of the concrete, is a very serious mistake. I think that you have to have moisture on the surface of the concrete. That is why we cure with moisture. Our surface of the concrete is warm to the touch and it is completely saturated. There is water actually all over the surface. We have very little trouble with surface thinning and cracks. The material looks good and it is soaking wet, which we feel is very advantageous. Do you intend, in your curing operation, to apply any moisture to the surface of your concrete?

QUESTION:

Yes, we have an all-steel bed form sitting on top of the bed. We are going to insulate the sides and apply heat and have a little waterspray system.

MR. VERNA:

I think that would be satisfactory as long as you maintain the moisture on the surface. I would suggest that you preheat the sprinkling water. In this way, you will not be counteracting the temperature rise that you are trying to get from your hot oil. I think temperature is very important in the early strength. We get, consistently, strength of 4,000 pounds and 5,000 pounds in 18 hours. We use eight sacks of Type iII cement.

QUESTION:

Is that with lightweight aggregates or heavyweight aggregates?

MR. VERNA:

In our operation we have, for the past year, been manufacturing and using in our secondary members, which are our double-T members, a concrete that only weighs 115 pounds per cubic foot. It is made with heavyweight sand and with an expanded shale of lightweight material made by a Carolina corporation about 40 miles from our plant. The reason we do not use 100 per cent lightweight aggregates is that in our ready-mix operation we know that since it is a porous-type of aggregate it takes considerably more mixing water to make a workable lightweight mix. We decided to try sand, which was not porous, and get our fines for our workability from that and just use the coarse and thoroughly saturate it. We found that we could maintain and make a mix with no more mixing water than we had in our heavy aggregate. The total water is 58 gallons. We get the nominal strength and it works better. We get a better finish out of it and an excellent elasticity.

QUESTION:

Mr. Rice, how do you handle your method?

MR. RICE:

Much of our work is actually a field operation. It is precasting that we are familiar with. We use truck cranes primarily in the yard. We have truck cranes available close to the yard, either owned by the corporation or on a rental basis, when on a large operation. It is not a good operation, and we are the first people to admit it. The first operation I had seen was Mr. Gerwick's operation with a rail crane. Perhaps, I could get that from Mr. Gerwick. He has a crane on rails that is very stable and fast. It is much faster than a truck crane and I think even comes pretty close to an overhead crane.

MR. GERWICK:

I might qualify that statement by saying this: Most of the units that we build are very long and very heavy. We are specializing in piles and long girders. For that reason, we often have to handle them with two-pickup points. It could be done with two overhead cranes. We also have to load them on a barge. Most of our transportation is by water. That is the reason we selected the locomotive cranes, which are faster and more flexible and are particularly adapted to loading on a barge.

QUESTION:

What did some of you gentlemen try to set up as standard in your area as to tolerances of precast members, and not in the prestress operations, in their lengths and widths? What about their physical properties for erection and, say, camber in commercial work? A lot of times we have had specifications come out where the architect specifies a certain amount of camber.

MR. VERNA:

If they make it too stringent, just bid them high.

MR. PRICE:

We have had some very rigid tolerances and we have had some reasonable tolerances. Yet, we do a great deal of design work of our own, that is, a complete precast, pre-assembled structure. The only time we get into trouble is when we detail it too closely. Every time we detail one, we give a little more tolerance than we had in the previous one.

In our own work, we like a minimum of a half an inch for the setting of a panel on a column or the adjustment of a column to a beam; or at least leave that much space if it is to be attached by welding. We are doing a job right now designed by Mr. Ed Rice that has a tolerance of 1/64th of an inch in it. I wish I had let my good friend, Mr. Gerwick, have the job.

MR. GERWICK:

I bid it high.

MR. VERNA:

On this question of tolerances, I would like to make one comment. We have had some sad experiences costwise in some of these non-standard units. In every case, it has been our opinion, after we completed the job, that at this stage of the game, prestressed concrete just was not the item to use for that job. I think we are at a critical stage in the development of prestressed concrete. We should not, as designing engineers, architects and manufacturers, attempt to use prestressed concrete where it is not applicable. Let us be cognizant of our tolerances so that we can practically make our lengths, our shapes and our sections with the materials that we have at hand today and let us not oversell the material because it will behoove all of us who are producers in the long run. We are interested in the long pull. We are not interested in what we can do tomorrow or the next day. It is a new industry, something that is beyond the comprehension of, I would say, 95 per cent of the engineers in the country.

MR. DEAN:

In this matter of tolerances in inspection, I wonder if you would tolerate a comment or two from me. In the matter of dimensional tolerances, I do not think there is any particular

difference between prestressed and other types of the precast type of member. In any such type of construction, there has to be some room for dimensional tolerance. It is extremely questionable whether you could always lay down exact rules for that and, I think, that would certainly depend upon the judgment in a large measure of the designer or, certainly, of the engineer who reviews the designer's plans. In regard to prestressing tolerances, I must say that I disagree with John Rundlett when he said he would be unwilling to specify any stressing tolerances. There is such difference in various materials that if you are going to issue a standard instruction to your inspectors, I think that a stressing tolerance is a requisite. Otherwise, you will have every inspector setting his own tolerances, which would be a very dangerous procedure.

In our work, using the benefit of our experience on the lower Tampa Bay Bridge and some of our observations in the early pretensioning yards, we have set a stressing tolerance of 5 per cent and have conveniently been able to stay within that limit. Our construction specifications require a measurement of a stressing force, both by jacking pressure and by elongation. In practically all of our stressing operations, we have been able to maintain our stressing error within that limit of tolerance. In fact, concerning most of our work in the pretensioning yard, I have read many, many of the reports and they fall within the range of two or three or four per cent. There have been a number of cases where the error was greater than five per cent, at which time they are supposed to stop and check up on things and find out what is wrong. Usually, they have found that there was something wrong.

On a recent job in one of our major yards here, they found they were getting the elongation before the jacks registered the proper pressure. A checkup of the bed found several of the cables twisted around each other. Had it not been for an established criterion for acceptance, the inspector might have accepted it and gone ahead with the job, which could have had disasterous results.

I was thinking about the requisites of a good inspector for prestressed work. It is certainly true that we have had to use some judgment in the selection of our inspectors. We certainly cannot take a pile hand and give him two or three rules of thumb and say, "Now, you are a prestressing inspector." That applies to lots of our work. We have tried our best to select our inspectors as men with some judgment and as much background as could be gotten. I also thought about the inspector who might make his beams with meticulous care, but then be rather sloppy in the preparation of his cylinders. The usual way would be just the other way around. There is no effort in making those cylinders come up to snuff and then they wonder what is wrong when the beam gets out of line. There is nothing in this inspection of prestressed concrete that is not susceptible to an analysis by ordinary common sense and good intelligence.

I do think that we are justified in selecting the best inspectors we can get for the work, but there is nothing to be afraid of as long as we ground those fellows in what they are doing, what they are getting after and then give them some reasonably uniform rules to work by.

MR. RUNDLETT:

I might say, Bill, that my subject, although it was not so announced, was: "Tolerances Within the Completed Structure." I will agree with you on the production of the structure.

QUESTION:

Mr. Price mentioned that he uses an English mixer, which I understand works on a counter-flow principle, and produces a concrete which is 15 per cent stronger than concrete made with an ordinary mixer. I have heard of those mixers before. They use them in Germany, too. Is there anyone here in the United States who makes those mixers? If it is 15 per cent better, we ought to make them here in the United States.

MR. PRICE:

I do not know of a manufacturer making a counter-flow mixer. I know they are made in Germany and they are also made in England. It consists of a drum about eight feet in diameter and two feet deep. It has, I believe, about eight fixed deflectors inside and it has two counter-rotating spiders with blades that agitate in counter-flow direction, somewhat similar to your wife's Mixmaster. When I spoke of a 15 per cent difference, I was referring to tests which we had made using a ready-mix truck as one type and a Besser paddle-type mixer as the other. The laboratory work is done with a paddle-type mixer and we compared test values of those two. We have the Besser type in the laboratory because we also manufacture concrete blocks and we use the Besser in the block manufacturing end of our business.

There could be a great deal of argument about there being an exact 15 per cent. If you were a manufacturer of drum-type mixers, there could be argument that there was any difference. Our experience has been very favorable, particularly that it will make and discharge a no-slump type of concrete.

MR. VERNA:

Mr. Price, how long does it take you to mix a batch in that mixer and what capacity is it?

MR. PRICE:

The mixer is a 50-cubic-foot mixer. It is rated at 30 seconds.

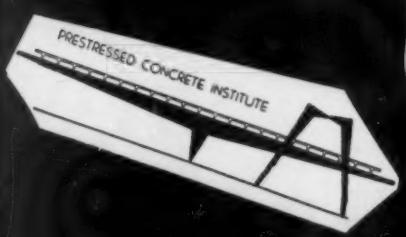
QUESTION:

What is the price of that mixer?

HURRY

Join Now!

The PRESTRESSED CONCRETE INSTITUTE is the clearing-house for all information related to the prestressing industry. Its publications and Short Courses, conventions and research programs keep members fully informed of latest developments in the design production, application, and installation of prestressed concrete structures. The one sure way to keep up with the progress being made by this dynamic young industry, is through membership in its national association——the PRESTRESSED CONCRETE INSTITUTE. Join.....TODAY!



Application Blank Enclosed

ROEBLING

FOREMOST SUPPLIER OF TENSIONING ELEMENTS FOR PRESTRESSED CONCRETE

Stress relieved strand for pre-tensioning.

•

Galvanized strand, assemblies and stress relieved wire for post-tensioning.

•

Write Construction Materials Division, John A. Roebling's Sons Corporation, Trenton 2, New Jersey.

ROEBLING

Subsidiary of The Colorado Fuel and Iron Corporation

MR. PRICE:

I believe it is somewheres in the neighborhood of $5,000.00 for that size.

QUESTION:

Can you get a full 50-cubic-foot mix?

MR. PRICE:

It will take 50-cubic feet, but it is hard to figure.

MR. HENRY NAGY:

Those mixers can be bought in Holland, Michigan. They have been made in this country for the last 20 years.

MR. VERNA:

What is their name?

MR. HENRY NAGY:

Dunn Manufacturing Company, Holland, Michigan; and there is a mixer that is sold in Canada by the Irish Mixer Company. You can buy them in Canada. It is a German make, but that is the only type of mixer that is used in Germany for concrete and I agree with Mr. Price that they are very good.

MR. VERNA:

Is that the type of mixer that your firm uses?

MR. HENRY NAGY:

No, we are not using them, but I have a couple of German engineers here and they claim they mix much better and I have seen them used over there.

QUESTION:

Is that the same as the Irish mixer?

MR. HENRY NAGY:

The Irish mixer has a pan about four or five feet in diameter and about two feet deep and you have rotating deflectors that go around. In the Dunn mixer, you have just one concentric mixer. I do not think the Dunn mixer is as good as the Irish mixer. The Irish mixer is the same as Mr. Price has. It is the same as the English mixer.

QUESTION:

I may not be very imaginative this morning, but perhaps, there are other gentlemen here besides myself who might be interested in a little more complete description. Is it a portable device or primarily a device for the manufacture of concrete block?

MR. PRICE:

The mixer is normally used on the continent for the mixing of all types of concrete. It is made in portable models and also in fixed models. Ours is mounted directly under our batching equipment. When we transport the concrete, we do it in a ready-mix plant. We use the ready-mix stuff only for transporting the material.

MR. VERNA:

That brings up a good question. How do you get no-slump concrete?

MR. PRICE:

You do not pour no-slump concrete on it.

MR. VERNA:

What do you haul your no-slump concrete in?

MR. PRICE:

With either a bucket, which we can transport with our lift truck. or with a horizontal drum mixing truck, which will discharge no-slump concrete into a conveyor.

MR. CONE:

With your experience, if you were going into business in a new area in conjunction with a ready-mix outfit, what length and what width bed would you recommend to someone going into this business?

MR. PRICE:

I will describe some beds that we have designed and have in production. We keep bounding around some lengths of 440 to 500 feet in our experience. We also seem to keep bounding around a bed in which you can cast two members, side by side. We built one bed in which you can cast three parallel and we found it is difficult to clean the tender bay. Those seem to be the only two big criteria that we have come up with so far. You have to find a bed where you have an intermediate support. We have supports, I believe, at 50, 100 and 200 feet and then the full length of the bed. I think you should have a bed in which you can apply harping forces. We think we foresee the need. We think it is a tragic mistake to build a bed and not put in the inserts.

MR. CONE:

What do you mean by harping?

MR. PRICE:

Pulling the wires down.

MR. VERNA:

It has been our experience that a 250-foot to 300-foot bed would be all that our company would ever put in for a single operation. It is our experience that when you get into longer beds time becomes quite a problem if you have 25, 30 or 45 people working for you. They can spend an awful lot of time walking that length of bed and not producing much. It is our opinion that several beds no longer than 300 feet, side by side, with some conveyor system so that your overhead crane does not have to do very much lengthwise movement utilizes

P. C. Items

Published monthly by

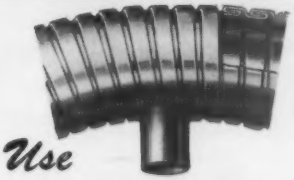
Prestressed Concrete Institute

This lively illustrated news bulletin presents news of Institute activities . . . views of members facilities . . . highlights outstanding prestressed concrete applications, and reports on latest developments in reserach, production of prestressed concrete.

Subscription $2.50 per year

To start your subscription, write to—

EDITOR
PCItems
3132 N.E. Ninth Street
Fort Lauderdale, Florida



Use
STEELFLEX
ALL METAL *Flexible*
CABLE CASING
For Armoring Post Tension Reinforcing Assemblies

*Lengths to your specifications Flexible Tubing furnished with cement grouting inlets if specified.

Write for Bulletin U-100-BX-1

Quality . . . ALL METAL FLEXIBLE HOSE PRODUCTS
UNIVERSAL METAL HOSE CO.
2133 S. KEDZIE AVE., CHICAGO, ILL.

your manpower more efficiently. I think that it is a mistake to build a 400 and 500-foot bed and have only one jacking end on it, because it takes manhours to get from one end to the other. I do not care how effective an organization is set up, there is always somebody who needs a screwdriver, wrench, trowel or something, and they are always walking back and forth. It spreads your operation out. We believe in more centralized control with not so much distance involved. If you want additional facilities, duplicate the beds further down the line and put another crew down there. You can operate very satisfactorily with several crews instead of one big crew. Break it down to small crews and stagger the crews. You may have one crew stretch out cables and have another crew come in four hours later and pour your operation, cover it and steam it. That is the direction we shall go.

QUESTION:

Are you unionized?

MR. VERNA:

No. We do not intend to.

MR. CONE:

What would you recommend to a person starting out from scratch as to what kind of bed to use in a new business?

MR. GERWICK:

My answer would have to depend on what you are going to make in that particular area. We have one bed 100 feet, one 300, another 340 and one 520 feet long. The 340-foot bed seems to be the most popular for the average run of products, but when we get into long piles, we need the 520-foot bed. The question on our minds is if we had made it 600 feet long, would it not have been better?

MR. VERNA:

I think it is a mistake to try to build a reversible bed to start out with. I think the important thing for anyone going into business for God's sake, analyze your market. See what you can sell and don't try to do a lot of non-standard things to start with. If there is a market for one story buildings and for warehouses and commercial construction, set up a bed to sell that material. Don't try to do a lot of different things. If you have enough business to have five beds busy, I think it would be to your advantage to have different beds.

Second Annual Prestressed Concrete Institute Convention

PANEL DISCUSSION

Design

CHAIRMAN

W. E. Dean, Chief Bridge Engineer, State Road Dept., Tallahassee, Fla.

PANEL MEMBERS

Paul Zia, Civil Engineering Dept., University of Florida, Gainesville, Fla.

T. Y. Lin, University of California, Berkeley, Cal.

Thor Germundsson Portland Cement Association, Chicago, Ill.

Eugene Smith, Freyssinet Company, New York, N. Y.

MR. DEAN:

In design, there are a number of considerations that are quite new to those of us in structural practice. I say, "new" and yet there is nothing illogical or out of the way in them. In every technical college in the United States, the science of structural engineering is based on certain assumptions: stress is proportional to strain, the distribution of stress is linear, and that plane sections before bending remain plane after bending. Those principles are entirely adequate for a structure designed of rolled steel shapes, with a limiting axial stress of let us say 18,000 psi. We can accept those without question. We can extend the elastic theory to the design of prestressed concrete structures. With certain modifications, it will be found expedient and necessary. Professor Lin, in the previous panel, stated that in his opinion the proper approach to prestressed concrete design is by the ultimate theory with a check of stresses at ordinary working loads. For most of us in structural practice, our planned approach is by the elastic theory with a check, as outlined in the Bureau of Public Roads' *Criteria*, for ultimate strength. It is probable that within the next 10 years, with a greater understanding of design practices, our current practice may be modified.

However, we will certainly not be able to get into that subject very deeply this afternoon. Our first speaker, Mr. Germundsson, is going to summarize several papers on prestress work which were recently given at the meeting of the Highway Research Board in Washington during January 1956. These papers are current, and Mr. Germundsson has digested and condensed them and will give us their high spots.

MR. GERMUNDSSON:

There were five papers given at the Highway Research Board that I will deal with briefly today. The first one was Bill Dean's paper. However, I am going to make that extremely brief because I think he will give it more treatment on Friday.

Here in Florida, of course, the trestle type of bridge is the prevailing type and that is where the prestressed concrete has been used the most. Mr. Dean stated in his paper that until recently reinforced concrete T-beams or I-beams with concrete slab built compositely were used, but prestressed concrete is now replacing both of them. The replacement is based solely on contract cost competition. Up to date, he said in his paper, the prestressed alternate has won on every job in which it was considered justifiable to be included. That was so consistent that the policy of offering alternate types has been abandoned.

Here is an interesting one that comes from the other end of the country, namely, California, Division of Highways. The paper was prepared by Arthur L. Elliott, Bridge Engineer for Planning. He stated that since 1950 when they built the Aroyo Seco bridge, which was one of the first in the country,

they have actually finished 20 projects in the State of California and 20 more ready to go.

He divided the 20 completed projects into several groups. There were seven in the first group. Those seven were chosen for prestressed concrete in order that the beams could be erected over water or traffic without intereference at all with what was going on underneath. In this case, it was economy combined with the elimination of false work.

The next group contained six bridges and there they chose prestressed concrete to reduce the depth of construction and allow for maximum clearance. In the next three they chose prestressed concrete because of the remote location. It was more economical to ship them over long distances, rather than to carry labor and the material to remote locations. In three others there were savings possible by the mass production of large numbers of similar types of similar cross-sections. In the 20, there was one unusual case which I will just briefly mention. This was a bridge that had to be loaded with three feet of earth for which it had not been designed. Therefore, the bridge girders had to be strengthened. They put girders underneath and post-tensioned them, thereby creating the camber that we are discussing these days; and that camber assured the distribution of the load from the old structure to the new, so the two cooperated thoroughly.

There you have some of the important reasons for using prestressed concrete. i.e., erection, reduced depth, remote locations, mass production and strengthening.

Mr. Paxton from Oregon, Assistant State Highway Bridge Engineer, said they built several structures of prestressed concrete at a cost that was rather erratic because the spans are only part of the structure. There has been some unbalance in the bids, and contractors still have too little experience with prestressed concrete to properly estimate the cost of their work. He thinks, however, from the scattered cost data, (even in 120 to 150-foot span range) prestressed concrete is competitive with steel. That is, of course, under the conditions prevailing in Oregon. That may not necessarily prevail in Illinois where the Gary steel works are so close by. His conclusion is that we have had enough experience to convince us that prestressed concrete has a definite place in our work. At present, its most economical field is in the short span pretensioned beams made in the manufacturing plants. For spans ranging from 80 to 200 feet the beams would have to be post-tensioned. "We like the low maintenance cost, we like the appearance and, where conditions are right, we like the prices."

Mr. Rundlett, who is here today, started out by blaming Florida for sending its hurricanes up to New England. The storm of August, 1955, destroyed or badly damaged more than 200 small bridges and their replacement would amount to more than $9,000,000.00. The greater percentage of these were under 40 feet and a study indicated that pretensioned units with hollow cores were particularly suited for this span range. They decided to make a new design that would apply to all span ranges and all loading cases from H-20 to H20-S16. They had two depths, 17 and 21 inches. The sections were four feet wide with three hollow cores in each. The only thing that was changed was the amount of pretensioning.

He gave some prices that are rather interesting. They let their contract in two groups and the second group came in six per cent less than the first. The cost data were extremely encouraging for both. The square foot cost in the 20 to 30-foot range was about $5.50. For the 30 to 40, about a little more than $6.00 per square foot, and two-thirds of that cost was for manufacture and delivery to a central spot. The other third was for transportation to the bridge site and erection. So, the total cost of these bridges erected would be somewhere in the neighborhood of $550,000.00 to $625,000.00. Approximately ten per cent of the total cost was for the prestressing strands.

Mr. Hogan, Consulting Engineer for the Portland Cement Association, took as his main subject: The Garden State Parkway, New Jersey. There were 12 bridges included in the bids received in October, 1953. There were alternates, and the contractor bid low on prestressed concrete. Bridges ranged from 40 to 60 feet. They used the same cross-section for all the girders and only varied the spacing and the amount of pretensioning steel in order to fit the desired spans. The superstructure cost $9.00 per square foot of which the beam cost was $5.00 per square foot. In October, 1954, they received, in New Jersey, bids on a harbor bridge in which prestressed concrete and steel were in competition. The low bid was just about the same for both and prestressed concrete was chosen because of its lower maintenance. The structure had 168 prestressed concrete beams which were 40 feet long at a cost of $16.00 per linear foot. The whole superstructure was $6.55 per square foot.

Mr. Hogan ended his comments by saying that in the course of five years only prestressed concrete has demonstrated its potentialities for bridge construction under all kinds of conditions in practically all parts of the country. The possibilities for future progress are bright and promising.

MR. DEAN:

Those papers that Mr. Germundsson mentioned, will all be published in the Proceedings of the Highway Research Board and will be forthcoming sometime this year.

Our next speaker this afternoon will be Professor Paul Zia of the University of Florida who is going to discuss the use of certain design concepts in standard building construction.

MR. ZIA:

What I would like to talk about for a few minutes this afternoon is some design considerations in buildings using standard prestressed concrete units. Standard prestressed concrete units are being used increasingly in various types of buildings, particularly as a roof. I would like to mention some design considerations as affected by the characteristics of these units.

Camber and the growth of camber are inherited as characteristics of prestressed concrete. We can control camber, but we cannot eliminate camber. In fact, we prefer to have some camber in our member to offset the deflection due to live loads. Due to this characteristic, it is preferred to place the prestressed concrete units whenever possible along the short direction of a building to avoid the unpleasing sight of the long wavy eaves line. In such arrangements, it will usually require a lesser amount of supporting members such as beams and columns. However, if it is necessary to place the units otherwise, then the designers certainly will have to be careful in planning the facade in order to cover up the long wavy eaves lines. Sometimes, it is also necessary to pour a thin layer of insulation or concrete to level the roof and avoid or eliminate any valley or pocket that may form on top.

There are, of course, the loading tables prepared for all these standard units from which a designer can select the proper member for his particular need. However, a designer and a builder sometimes misuse these tables because they overlook the characteristics of these members. One example is this: Suppose a building has two spans of 20 feet each. The designer or builder may select a member from the table suitable for the 20-foot span and loading. Instead, he casts a 40-foot member with the idea that the member will crack when it rests on the center point and so he would have two simple beams. However, he does not realize that the camber may prevent it from resting at the center support. The result is some overload on the outer supports.

As a second example, a building may have a 40-foot span and also a 30-foot span; and if the loading in a 30-foot span is so big—the moment in the 30-foot span is greater than that of the 40-foot span—then, the designer may select a proper beam from the table, so that it will be suitable for both with the idea that he is on the safe side. However, the result is over-designing. The 40-foot span member does give excessive camber.

The third consideration is the growth of the camber. The growth of camber is, as you know, due to the creep and the temperature change. I have often been asked the question how much does this 40-foot double-T expand when temperature changes. There is no definite answer that can be given to this question. It certainly is a good point for research, but for practical purposes, one seems to be able to estimate this growth in the camber due to temperature change in this following way: When temperature increases, the top fiber of the member will elongate, but the bottom fiber will not. This will cause a difference in length between the top fiber and the bottom fiber and this difference can be considered as though introduced by an equivalent moment applied at the ends of the member. The upward deflection for the simply supported beam due to this equivalent moment can be determined by the usual deflection formula.

On this basis, it is interesting to note that the growth in the camber due to temperature change seems to be proportional to the square of the span and inversely proportional to the depth of the member. It is independent of the modulus of elasticity and the moment of inertia of the section. From this reasoning, with the temperature increase of about 50° F, a 40-foot-span, double-T, probably will have an inch movement upwards and a 60-foot double-T will have about an inch and a half movement. This seems to agree with the reports from the casting yards fairly closely. With this kind of magnitude in movement, it is obvious that the designer will have to be careful in the type of ceiling. If there is any partition wall at all, it should not be rigidly attached to the roof. The roof should not keel into the parapet, if there is any.

To conclude, I would like to say this: The prestressed concrete certainly is not a cure for all. It has its advantages and its shortcomings, but if due considerations are given to the shortcomings and to its characteristics, prestressed concrete will give us strength, economy, and good appearance in our building.

MR. DEAN:

One field in which most of us who are working in prestressed concrete have been remiss is that of continuity. There have been some very notable exceptions to that. Ross Bryan, in some of his building work, is one. There have been others. There is by no means agreement among designers in prestressed concrete as to the value of continuity, in small structures at least. The larger the structure gets, certainly the greater value of continuity.

We have with us this afternoon Mr. Smith of the Freyssinet Company, who is going to give us some comments on continuity in prestressed design.

MR. SMITH:

I would certainly have to agree with the comment that Mr. Dean just made that we have not been applying continuity in the United States. There are some bridges and some buildings, but there is no general application of this particular construction technique. The continuity itself can offer considerable savings in construction. This is offset by natural disadvantages in attempting to achieve continuity, but the ultimate saving and the ultimate architectural effect that is possible by working with a shallower section is undoubtedly going to force us into it—in one way or another at some time

or another.

Now, the general disadvantages of continuity: the first obvious one would be that you are working with a more complicated design. It is not only in the members themselves, but it is going to affect whatever supporting members you have and it can affect the foundation of the buildings. When we go into continuity, there are a lot more factors that will have to be considered and a lot more thought has to be given to the actual layout of the structure.

On the job site itself, with the actual erecting and assembling of a structure, there are complications which can arise strictly due to the introduction of continuity. The use of mild steel and continuity is a relatively easy technique. Most of the other ones, which are probably more structural sounding, become more complicated. Perhaps it would be a good idea to go through techniques for achieving continuity that have been used.

Some have used cast in place sections which were made continuous by the introduction of a cable or a post-tensioning unit that follows the requirement for the selection and the moments for each section of the beam. This introduces a somewhat wavy line in the prestress tensioning itself. In general, it is going to force the cost up because it is insisting upon a site construction principle. Now, in this particular type of continuity, the next factor entering into it is that when you apply the prestressing force, you are going to directly affect the moments within the member itself, and the reactions at the supports. These all have to be taken into consideration and the design becomes somewhat more involved, although on a fair-sized structure, it is not prohibitively so.

To get away from the possibility or the necessity of having prestressing tendons flow up and down with the various moment sections, they have used haunched members. This would permit the prestressing tendon to be essentially straight or reasonably straight. Each of those techniques would only be applicable, I think, to a good-sized structure. It is certainly not so in the ordinary building work, but it may be in the long-span bridges.

To achieve continuity at all, economically, you are just about forced into some proposition of precasting and assembly work on the job. Now, precast members can be pretensioned for dead load and put in place in their final location in the structure and some other means may be used for achieving continuity over the pile caps or girders or whatever the supporting members may happen to be. These beams on fair-sized jobs again can have their continuity assured by the use of so-called cap cables. These are post-tensioned units that are located at the supports where you want the negative moment developed. They will extend through the beam, up across the top and then out the bottom and outside of the main support. Theoretically, this is a very fine system. Practically, it is a pain in the neck. It just is not ordinarily feasible or economical to get men under a bridge member to do all this tensioning and it is not ordinarily easy to collect all the high tensile steel in that section of the member.

The system that has been used probably the most in the United States has been by the use of mild steel to develop continuity. The members are precast and prestressed for dead-load only. They are put in place with mild steel and dowelled into the precast member. The steel bars are extended over the support to be enclosed in the poured concrete, thereby establishing a degree, at least, of continuity over that particular support.

Now, on this setup, there is a lot of controversy on just how effective that mild steel is going to be. The main part of the member, or the part which is resisting the positive moment being prestressed, is going to perform elastically as it should for all ordinary loads. However, the part over the support being reinforced with mild steel is going to have to crack before the working stress can be developed in the mild steel. The cracks at that particular section are going to introduce something approaching a hinge. You lose the moment at the support and more positive moment is thrown into the midspan.

When a design is made, and if recognized it can be accounted for, you can probably get continuity for a reasonably priced structure. Now, the continuity certainly has to be introduced, as I mentioned before, because we always have architects who want shallow sections and highway engineers who would like more clearance under the bridges.

All of these processes that have been mentioned are expensive one way or another, except, perhaps, the case of mild steel. The mild steel continuity proposition has been used in buildings. The other techniques are applicable only to pretty heavy structures where you can afford to do some special work. My opinion is that most of our continuity is going to be achieved by the mild steel. For a specialized system or for a larger structure, the mild steel will not be the answer, you will go to draped cable or haunched members.

MR. DEAN:

The subject of continuity looks rather complicated to us in prestressed design at this time. I have no doubt that suitable techniques will be developed.

Our next speaker is Professor T. Y. Lin of the University of California. As has been mentioned several times, Professor Lin is the author of one of the most recent and authoritative works on the subject of prestressed concrete. One of my most treasured possessions is a complimentary copy of that text. I read a good part of it in detail and I have skimmed all of it. I will not pretend that I have absorbed everything in that book. I probably never will. Certainly one of the most interesting things in that book is his dedication: "To engineers who

rather than blindly follow the codes of practice, seek to apply the laws of nature." I hope that I can be numbered among that group and I believe that expresses a man's philosophy of design. Certainly, throughout the book, reading between the lines, you can find that philosophy cropping up from time to time. Professor Lin is going to continue the comments on continuity. He will also give us some comments on camber control. If he feels inclined to intersperse any of his philosophy of design in his comments, I trust he will feel free to do so.

PROFESSOR T. Y. LIN:

For all the free advertising you have given my book, you certainly belong to those engineers to whom the book is dedicated and I hope very soon the whole audience will also belong to that group.

A while ago we were talking about stresses. Stress is something so unreal. It is much easier for us to talk about it. Nobody knows what it is. When you talk about camber, it is something you can see with your eyes. There is no prestressed concrete member that you can design with absolutely no camber. It is only possible to design a member that will be level under a given condition. Then, if you put a load on it, it will deflect. Under another condition it is pretty nearly level. However, with the unknown properties of concrete and steel, we cannot make it exact.

I will start first from the point of designing. It is possible to design a slab or a beam so that under certain conditions with a given load it will be perfectly level. The entire trick is to have the distribution in the whole section of the beam or the slab under that given condition. In other words, if your center of force passes right through the center of your concrete, according to what Bill just said, the plane section remains plane. You have to know what the real loads are and not the fictitious loads given by any code. What is the real load that member is going to carry most of the time? Of course, this is merely a guesss oftentimes. You do not know in warehouses how much they are going to store, so you have to guess. But the best guess you can make is the load expected to be carried most of the time and design for that particular load so that it will remain perfectly level.

There are some other questions. It may be level before it is shipped, and then when you ship it from the pretensioning yard, it will camber upward. That is a big problem. Three years ago in England I visited a pretensioning yard that had gotten into trouble. They shipped all the members to the site only to find that they cambered up too much. After storing it for months, the camber grows and grows. So, you have to think about that, too.

Now, you cannot have everything. You cannot have it exactly level with no load and exactly level under full load. It is possible, for example, to have it very nearly level under no load, to put it into the building, jack it up, prop it up and then put the cover on, or whatever additional floor you may have, so that you can remove the problem. You cannot always do that. So, there you have to choose between the devil and the deep blue sea. You can say, "Well, I will design it under this condition." It will camber up and in the other condition, it will deflect down, but not too much. That you can do by proper choice of your concrete sections. Take a double-T section, for example. Under dead-load, you put your wire so that your line will go pretty nearly through the center of that section. You cannot always get it exactly. Take an I-section. That will be favorable. Take a rectangular section. That is better for camber control, although generally it is not economical.

What other things can you do to control camber? You can vary the amount of prestress so it will not camber too much. You can change your prestress by greasing near the end of the member so it does not get as much moment. You can bend your wires up and down in pretensioning or posttensioning. That is one way to control your camber. You can use all of these techniques to get just the right amount.

Then comes the question: After you control all of this, if you do not know your concrete well enough, how can you predict the amount of camber? That I think is difficult, but you can minimize your unknowns. The whole trick, I think, is to minimize your unknowns. Someone mentioned a while ago, that prestressing with mild steel would not work because of the amount of the loss. Instead of the 100 per cent prestress, you reduce it by 15 per cent. If you are wrong by 15 per cent, you would not probably be wrong by 10 per cent or 20 per cent. You are not very far off. If it comes at 10 per cent, you are only five per cent off. So, there is this trick which is in part inherent to prestressed concrete and that is reducing your unknown values to a small fraction of the total value.

It is very interesting when we balance one unknown against the other. For example, creep causes the camber to grow, but the stress loss helps to reduce creep and tends to decrease camber. If your sections are designed properly, the relation of dead-load and live-load is favorable, then you can obtain a proper balance of one unknown against the other.

QUESTION:

Mention was made about the various considerations on the ultimate load design. Well, I happen to be on the ACI-ASCE Committee on factor of safety. I wonder if there are any characteristics of prestressed design which would change our factor of safety, our consideration of our factor of safety. In other words, what is the factor of safety we should use, the same as we use in the ultimate load design?

MR. GERMUNDSSON:

Mr. Ruble, on Friday I am going to show a slide on which I have chosen from an English publication three instances of comparison between ultimate strength design

and service load design. Two of these will be extremes and one will be an average; and, there is something about prestressed concrete that is definitely different from conventional reinforced concrete.

MR. DEAN:

I believe that some light might be thrown on your question by the statement that prestressed design, as I see it, throws one more step in the design practice, one that we have not been accustomed to using.

Now, if we are designing a rolled steel section at 18,000 psi, this stress is about less than two-thirds of the yield strength and about one-fourth of the ultimate strength. We can overlook some secondary stresses. It would be almost impossible for us to design an unsafe structure. When we are working with the stresses that we use in our elastic approach to prestress design, we are using much higher percentages of the ultimate for both steel and concrete than we would ordinarily use in our regular practice. For that reason, if we approach a design in the elastic manner, as we have all been accustomed to doing, we have this final step of checking for the ultimate load. In my own office, the ultimate requirements given in the Bureau of Public Roads' *Criteria* have influenced a number of designs. We have designed members which were perfectly adequate for the working stresses allowed. Then we checked the ultimate and found that they were too small. So, we had to go back and revise the design. It is just something that has been added to our formal design process.

MR. GERWICK:

I would like to ask Mr. Smith, with regard to his discussion on continuity, if he has any comments on the use of prestressing to join, say, a girder to a column to develop a fixed connection?

MR. SMITH:

Well, the exact trajectory of the cable can have a considerable effect upon the moment you may be introducing into that member. Now, in general, if you are going to achieve continuity structurally, the best way to do it would be by prestressing in one form or another. I think, it is far superior from the structural standpoint than the use of mild steel, although it may not be as economical. I would have nothing against joining the girder to the column. It depends on how the particular prestressing tendon is arranged and that should be definitely taken into consideration.

MR. GERMUNDSSON:

May I add to that? Mr. Gerwick would, undoubtedly, be interested in reading the article that will appear soon in the ACI Journal, describing one of the longest prestressed spans ever built in this country. It has an overall length of 145 feet. It is located in Springfield, Missouri. There, they built a column in two parts with a slot in between. They cast a girder on the ground and then lifted it in place. It was simply supported at that stage. After the roof was on, there was a way provided for prestressing the girder into the two parts of the column, so it attained full continuity for live-load and it worked out very well. The prestressing tendons were very short. Therefore, the elongation had to be controlled very accurately.

MR. SMITH:

I am sorry, Mr. Gerwick, were you actually talking about a framed continuity?

MR. GERWICK:

Yes. I am talking about girders into the column.

MR. SMITH:

That has been done on several jobs here in the States already, you know. Some examples are the Queensboro North High School, Northwest Senior High School, and one or two others, where it has been done for the existing live-loads.

MR. ROSS BRYAN:

I would like to comment on the connection of a rigid frame at the knee, which we did about a year ago, having a 65-foot span. We precast the vertical leg, conventionally reinforced, and anchored conventional reinforcing steel in the ends of prestressed, precast girders and welded that to the column steel. That was made continuous for live-load with the exception of the dead-load of the girders. That worked satisfactorily and made a very good job. However, in connection with that, I want to correct an impression that, I believe, has been left that continuity is something that has not been done and that it is untried. We have been designing all of our buildings and bridges, which have more than one span, as continuous members and that continuity has been obtained with mild steel. We have literally hundreds of buildings completed which are all continuous under partial dead and full live-load or any variations of that. Those members are made out of precast multi-unit block beams or they are made out of pretensioned girders. They are made in a number of ways and under many conditions. We have never had a bit of trouble. Many of those buildings have been overloaded in extreme cases and they have all performed very well. The basis of our design was established more than three years ago. We made some at the National Rico Plant on a small three-span continuous multi-unit beam and we found that the mild steel performed very well. It would seem that the concrete might have to crack before it acts. but that is a misleading statement because you can say the same thing about reinforced concrete. However, you do not see the cracking in any of our continuous reinforced concrete structures. Neither do you see the cracking in any of our continuous structures in prestressed concrete.

Now, a very, very large full-scale test was made of that type of construction about three years ago for the Army. We made an alternate design for prestressed multi-unit joints involving three spans. Two complete setups were made and the assembly was tested to the satisfaction of the Corps of

Engineers. That was done by a company in Ohio. Gordon Lord is the Consultant of that company and he is here.

We were questioned originally as to the per cent of continuity which we could develop by making it continuous with mild steel over the support. At that time, they wanted us to assume about 50 per cent continuity which would affect the course of the design considerably. We agreed to run load tests for them jointly with the Corps of Engineers and, based on comparative deflections which were taken rather accurately, we showed we were developing 88 per cent of the full continuity – which is pretty high; and, based on overload, the yield would be such that actually you would get full continuity.

MR. VERNA:

I would like to continue the discussion of the use of mild steel in continuous girders for structures of a one-story nature, such as warehouses, where girders are cast separately with mild steel left at the top for, say, live-load and for dead-load of the roofing.

In the analysis work that we have done (we likewise have done a considerable number of structures that way) we find that in checking the ultimate load of the girders to arrive at a realistic value of how exactly you have welded the mild steel together and poured the concrete, we have to discard the continuity since at the ultimate load we have a simple beam again.

That is the nature of continuity. So, the balancing out of prestress required for the dead-load is such that we always have enough steel left over and we have large safety factors in the neighborhood of something like six to seven times live-load. But when we get beyond the working load, there has to be some failure of the welded connections. So, the failure of welded steel takes place. As long as the member is in the overloaded condition in its deflected state it does not deflect seriously; that really is not a problem. Actually, we do it just to tie the structure together and give it stability.

MR. DEAN:

If I understand you, then, what you seem to be conveying is that you make your structures continuous by construction and that until such time as a load is applied, that the steel actually ruptured in the top of the member, you have a continuous structure for live-load; and there is little likelihood of such a load being applied, and if it were applied, it might just apply by breaking down the prestressed portion in the mid-span.

MR. VERNA:

No, because you have too much steel left over. In our structures, we have 35 by 40, 35 by 50, the negative steel reinforcing really performs the function of tying the structure together.

MR. SMITH:

On Mr. Verna's proposition of mild steel developing a hinge, that strictly is a design consideration. You might just as well have the plastic limit reached so that you would have a hinge at the mid-span first and later develop a hinge at the support.

Now, concerning what Mr. Ross Bryan was saying, it would seem that at least the tests have indicated one thing I said to be true. I frankly did not know about those particular tests, that 85 per cent of the continuity was developed; now, that is not a fully continuous structure. I assume that was under live-load.

MR. BRYAN:

Live-load.

MR. SMITH:

It was not a fully continuous structure. The possibility of developing a hinge at that point, at the support, is definitely present even though you developed 85 per cent continuity.

MR. BRYAN:

That was considered in the design. I would like to point out that the 88 per cent did not mean a thing. It means just exactly what your cracking load might mean. It is an unimportant thing. It was based on the calculated deflection of that beam.

Now, when you can calculate the deflection of two separate beams and carry them within 12 per cent of each other in deflection in a beam made up of composite construction, maybe it was 88 per cent and maybe 108 per cent. I mean, the tests are not accurate enough to determine that. The important thing is that under failure in that test and other tests we have made, 100 per cent of continuity was developed. In other words, the beams carried the full calculated ultimate load and, as far as I am concerned, whether they deflected within 12 per cent of the calculated value, I think, is unimportant. It is also unimportant whether or not you are within 12 per cent of the cracking load. It does not affect your ultimate load.

MR. SMITH:

I would not like to give the attitude that I am against mild steel continuity. I think it is the only economical way we have for ordinary structures. When you are depending on design, you must definitely consider the possibility of developing that hinge. Now, one section, of course, can develop a full one hundred per cent. You can get into trouble with other sections if they are not properly designed.

Second Annual Prestressed Concrete Institute Convention

PANEL DISCUSSION

Materials and Techniques

CHAIRMAN

R. M. DuBois, Freyssinet Company, New York, N. Y.

PANEL MEMBERS

H. J. Godfrey, John A. Roebling's Sons Corp., Trenton, N. J.

R. O. Kasten, Union Wire Rope Corp., Kansas City, Mo.

Ernst Gruenwald, Lone Star Cement Company, New York, N. Y.

Nathan Levine, Elgood Materials Corp., Brooklyn, N. Y.

A. H. Gustafferro, Portland Cement Association, Chicago, Ill.

Alvin Schwab, United States Steel Corp., New York, N. Y.

MR. GRUENWALD:

First, I have to apologize that I do not have a prepared paper. I understood this is to be a panel discussion. And, it was to be informal and the interest was to be developed by the discussion from the floor. I have made some notes, and I will talk from these.

In prestressed concrete, we deal with high quality materials. This is summarized in the works of the Joint Committee of the ACI and ASCE, where they say: "The nature and economics of prestressed concretes require the use of high-strength materials capable of safely resisting sustained high stresses with a minimum of time-dependent change in stress or strain."

Then there are certain properties of concrete which are of particular importance in prestressing work. These are: high compressive strength, high modulus of elasticity and low drying shrinkage.

My remarks will concern, primarily, cements and their effect on producing a high strength concrete.

In general, the cements and all the other materials are to meet the specifications of the American Society for Testing Materials so that they can be used with confidence. These specifications are the result of the cooperative work of users and producers and general interest. Naturally, there is quite a latitude, but from the overall standpoint, these materials will give satisfactory results.

In the ASTM specifications, we have five different types of cement. The Type IV cement was the type that was used in dam construction and has a low rate of hardening. However, I believe that has been eliminated from the specifications.

Type V cement is a sulphate resistant cement. It is not generally commercially available.

For prestressing, therefore, you will have a choice among Type I, Type II, or Type III cement, and their air entraining companions which have the same general limitations. As applied to prestressing, the most important difference between the Type I, Type II and Type III cements is the rate of hardening.

For economical reasons, it is necessary to get the required strength for prestressing at the earliest possible time. That will, in a way, dictate which type you are going to use. The time at which the strength is attained is dictated, I would say, by the operational setup of your plant. Three or four thousand pounds per square inch strength can be obtained in 18 to 48 hours by Type I cements easily if you apply heat to it and accelerate the hardening process. With a Type III cement, it is quite possible to attain that strength at normal temperatures.

The same fundamental factors affecting our normal construction concrete apply naturally to the high strength concretes, and the water cement ratio is the critical factor that will dictate or govern the selection of materials to attain this high strength at early ages.

You will hear some fellows talk about a six and a half bag mix and others talk of an eight bag mix and others talk of a ten

bag mix in order to gain these high strengths. The reason for that wide variation in cement content lies, of course, in the choice of aggregate sizes. If you use lightweight aggregates, the cement factor will be higher, in general, than it would be in the heavy aggregates.

The other factor of importance is the degree of workability that your operations permit. In general, it will be found that you want low slump concrete for reasons of having a minimum shrinkage. The selection of individual ingredients for your concrete mix will depend entirely upon the objective, which is the high strength at a given time and the low shrinkage. With high strengths, you also get a high modulus of elasticity.

The modulus of elasticity for the same concrete will not be identical. That depends on the type of aggregate you use. I don't think I am telling you anything new, but oftentimes we speak glibly of say, a 4,000,000 psi modulus of elasticity, disregarding the size of the aggregates. Now, when you use small aggregate, you are using more and more for the mix, and the modulus of elasticity will be lower for that particular type of concrete.

Because of the interest of the prestressed concrete manufacturers in attaining early strengths, we ran a series of tests in our laboratories where we tried to duplicate field conditions. We used two mixes, one with 5.7 gallons of water per bag and another one with 4.75 gallons. The sand had a finess modulus of 2.57. The gravel was graded from number eight to three-quarter inch and we used 60 per cent of the larger size and 40 per cent of the smaller size and had a combined finess modulus of 6.50. The cement factor was such that three inch slump was obtained.

One set of cylinders were moist cured at 73 degrees Farenheit. We wanted to find out whether the heat curing would in any way affect the later strengths. Another set of cylinders were cured in a manner that is often practiced in the prestressed concrete plants in applying heat after four hours of normal temperatures to about 140 degrees Fahrenheit for 14 hours. Now, we know concrete should be cured, but oftentimes it is not; so, one set of cylinders were exposed to the laboratory air immediately after the 18 hours of pre-curing, while another set was cured for three days—which, we thought, is about the practical limit. Then we tried another set where we pre-cured for eight hours or steam cured or hot cured for only ten hours, so that we would gain ten hours curing period. The latter, incidentally, gave the least favorable results. You might be interested in this six and a half bag mix, using Type III cement cured at 73 degree temperature. We attained strengths of 2300 psi in 18 hours and 2700 psi in 24 hours, and it took three days to get 4,000 psi with this six and a half bag and 5.7 gallon mix.

We increased the early strengths by increasing the cement content or reducing the water ratio,. and when we used 4.75 gallons per bag, which required an eight bag mix, at 18 hours the strengths increased from 2300 to 3,000 psi. All these concretes gave over 5,000 psi in 28 days.

You might be interested in this: When we exposed that concrete to air of the normal relative humidity which ran around 65 per cent, the strength of moist cured cylinders were 5,840 psi and the air cured were 5,280 psi, a loss of 600 psi.

The richer mix, strangely enough, lost practically nothing: 6,300 psi in 28 days and 6,280 psi for moist cured cylinders.

When we applied heat to this lean mix, the 14 hour strength jumped from 2,300 to 3,700 psi in 18 hours; and in 24 hours we had 4,200 psi. With the rich mix (the eight bag mix) we got 4,900 psi in 18 hours.

Now, I do not say that all these strengths are typical, because they do depend somewhat upon the aggregate, but with these low water ratios in Type III cements and with heat curing, you can get the required prestressing strengths in 18 to 24 hours, but never later than, say, 48 hours.

The heat curing did not seem to affect the 28 day strengths. When we exposed these cylinders to air after 18 hours, the leaner mix produced 5,640 psi in 28 days as compared to the standard moist cured cylinders of 5,840 psi.

On the richer mix, we got slightly higher strengths, actually. So, I think these tests gave us at least an indication or an assurance that we do not have to be afraid of applying the heat.

I would like to point out, however, that the concrete, when placed, had a normal temperature of about 76 degrees Fahrenheit and we kept that concrete at 73 degrees.

The question that is often raised: How about the volume change or the shrinkage upon drying out?

In this case the shrinkage was determined in a millionth of an inch per inch measured on a 3" x 3" x 12" beam, exposed to air of 60 per cent relative humidity. On the 6.5 bag mix, we got 590 after six months, and on the 8 bag mix, we got 555 after three months. Thus, the eight bag mix did not produce greater shrinkage.

MR. GUSTAFERRO:

Early this year ACI-ASCE, Joint Committee 323, prepared a preliminary report entitled "Recommended Practice for Prestressed Concrete." I am sure many of you have seen this report which was distributed to the 108 members of the committee for study. The report is not intended for use as a standard practice, rather it serves as a basis for discussion. And, the discussions are intended to be used in revision of the report.

I should like to review this report which contains four chapters; Chapter I, Introduction; Chapter II, Design; Chapter III, Materials; and, Chapter IV, Construction. I would like to limit my discussion to those portions of Chapters III and IV which deal with concrete. The sections on concrete as a material and its mixing, placing and

curing were inteded to serve as a guide for the production of high strength uniform quality concrete.

The points which I will talk about are those which I feel are somewhat controversial and could stand some justified discussion from this group. The first point that I would like to mention is slump. In order to insure a low shrinkage concrete, a low slump concrete is required.

The report recommends that one to four inch slump be used. It states that slump in excess of five inches should never be used for prestressed concrete. Those are the debatable values. My own personal opinion is that a four inch slump is pretty high. You are liable to get quite a bit of shrinkage with a four inch slump. I think, generally, you can get by with a much lower slump than that.

Secondly, the strength; the report does not recommend a minimum 28-day-strength, nor does it recommend any specific strength at the time of prestressing. Some engineers feel that this should have been specified or at least included in the recommended practice, while the majority felt that the strength of the concrete at the time of prestressing and at 28 days is merely an economic factor in the production of prestressing concrete.

The third point that I want to mention is the placing of concrete in level lifts. The report recommends that concrete be placed in level lifts from 6 to 16 inches deep. Some engineers think that concrete should not be placed in level lifts in prestressed concrete, rather it should be placed starting at both ends of the member and working towards the middle to the full depth of the section. They claim that the placing of concrete in horizontal lifts might produce planes of weakness at the lift lines, and that by placing the concrete continually from each end to the center any planes of weakness would be more or less perpendicular to the diagonal tension cracks which might occur at high shearing stresses.

The next point is concrete temperature. The report recommends that the temperature of concrete during placing, mixing and curing should remain between 50 and 90°F. unless special high temperature curing is used.

My personal opinion is that 90° is too warm for concrete. I feel that it should never exceed 70 or 75 degrees, and I think that that particular statement should be changed. The disadvantages of using concrete as warm as 90°F. far outweigh any possible advantages.

Another topic is the section on curing. The report contains only one paragraph on curing, and states that concrete should be protected against loss of moisture for at least seven days for normal Portland cement concrete, and three days for high early strength concrete. Longer periods are suggested for hot or cold weather. Shorter periods of protection are allowed for steam curing. However, I think that the subject of curing should be spelled out in great detail in specifications for any particular project.

Probably the only explosive issue that I will discuss is the subject of admixtures.

The report states that any admixture which is beneficial to the concrete quality may be used, provided that it can be shown that the admixture will have no deleterious effects. The report states clearly that calcium chloride should not be used in prestressing because of the possibility of corrosion of the prestressing steel.

In conclusion, I want to repeat that the intent of the committee was to recommend a practice for high strength uniform quality concrete. The comments of all of those who are interested in prestressed concrete will be studied and possibly incorporated into the final drafts of this particular report.

QUESTION:

Mr. Gruenwald, I would like to ask you two questions. You have emphasized the importance of heat. Are you thinking in terms of steam curing or heat curing through pipes imbedded in double T beds. If you use steam curing, you have the pipes on the outside; you actually have the effect of heat on the surface. I want to know if the heat penetrates way down to the bottom of a 16 inch, let us say, double T stem. The point where you actually want the highest strength is where you have the highest compression. If the pipes are located at the bottom you will have heat there where you want it, but how does it effect the outer surface if the outer surface is cold?

MR. GRUENWALD:

What I mean by heat curing primarily is the concrete gets the favorable temperatures at which the curing is accelerated. This happens whether you use steam or heat. The concrete gets warm and also you get the benefit of the heat of hydration.

The important part is that you cover the concrete and prevent the evaporation of water, otherwise, you would get into trouble. The cylinders that I talked about were put into a hot room and they were covered. I believe that due to the fusion of the heat it would be more effective to start from the bottom up rather than from the top down. But, that is a construction manufacturing problem. The important thing is that if you apply heat you have to prevent evaporation of the water.

And, probably everyone has had experience with these plastic shrinkage cracks which occur on top of the concrete when the water leaves the concrete too fast. It is immensely important during this hardening period that water stays in there. And, if you have a water type membrane, whether it is a cover or a membrane, it would not make much difference.

QUESTION:

Mr. Gustaferro, what is the practice regarding air entrainment in bridges, is it used or not?

MR. GUSTAFERRO:

I think that the introduction of air entrainment in prestressed concrete is merely a matter of obtaining the workability. I do not think that it is required for durability, because the high strength concrete that is used automatically would insure considerable durability. I do think that the entrainment of air for the purpose of obtaining a more plastic workable mix is beneficial. I cannot directly answer your question as to what is the practice. I think that maybe Mr. Dean or somebody else would be more qualified to answer that than I.

MR. GRUENWALD:

In the series of tests that I referred to awhile ago, we also used air entrainment cement. We did not get the high percentage of air but, just about double of what we got with the regular cement. The heat cured specimens practically showed the same early strengths as the later strengths. So, you do not lose strengths there. In the northern climates, where there is the problem of snow removal, they are using air entrainment cement just as an additional safeguard against loss of strength.

MR. GODFREY:

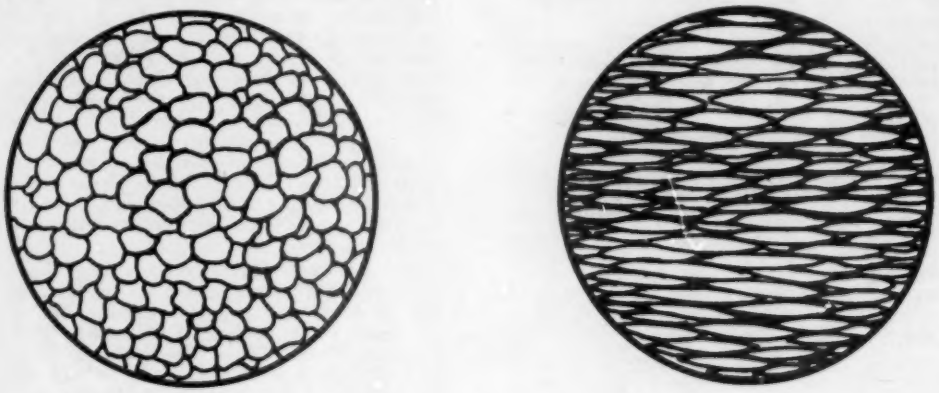
During the initial period of prestressed concrete construction in this country, there have been relatively few difficulties with its application, particularly when we consider its radical departure from the construction methods used in ordinary reinforced concrete.

However, there have been a few problems arising from the use of high strength prestressing steel with properties considerably different than those of reinforcing bars.

Reinforcing bars are manufactured by hot rolling steel billets which results in a product having a relatively low tensile strength. Steel wire used in prestressing concrete has a tensile strength of four to five times that of reinforcing bars and several manufacturing procedures are necessary to produce such a high tensile strength combined with ductility.

Wire is processed by hot rolling billets into what is known as a "rod." For uniformity and desired metallurgical structure, the rod is heat treated. The heat treated or "patented" rod is then cold drawn through a series of dies which produces the proper size and physical properties. The cold drawing operation results in a large increase in tensile strength and changes the heat treated granular structure to a cold drawn fibrous structure.

In Fig. 1, you see the heat treated structure—this is just a diagramatic view of what steel looks like when it is heat treated—you have grains of steel. Now, you still have the grains after cold drawing, but they become elongated and fibrous in nature. And, of course, the more wire drawing you do, the more fibrous it becomes. It is through this cold working that the material obtains its high strength and its toughness. After the wire has been cold drawn, it is then stress relieved to obtain high elastic properties and improve its straightness. This stress relieving temperature is not high enough, however, to change the fibrous nature of the material.



Godfrey Fig. 1 — Illustration of heat treated and cold drawn structure.

After the finished product has met all the required specifications and shipped to the customer, there are a number of factors which can influence the quality of the wire. These factors are corrosion, mechanical damage and heat.

The corrosion of wire and strand is not a particular problem under normal circumstances when materials are used within a short time after arriving at a job site. How-

Godfrey Fig. 2 — Longitudinal section through wire showing change in structure at point of fracture.

ever, wire or strand left unprotected for an extended period of time may become pitted to such an extent that its strength and ductility would be affected. The rate of corrosion will depend upon the conditions existing at each particular location and prestressing materials should be protected from the elements as necessary.

Prestressed concrete wire and strand are not generally subjected to any serious mechanical damage. Caution should be taken to prevent the section of the strand which has been in the pretensioning grips from being used in the structure. The sharp notches made by the grips will decrease the ductility of the wire.

The effect of heat on the properties of wire is not well known and a study of two such examples may be of interest.

One of our first investigations on the effect of heat was concerned with a wire which had failed during prestressing at a load less than the required pretensioning load. A test made on the actual wire in question indicated that the material was satisfactory in every respect.

A more detailed metallurgical study of the original fracture showed that a very short section of the wire had been subjected to a temperature of sufficient intensity to completely obliterate the cold drawn structure.

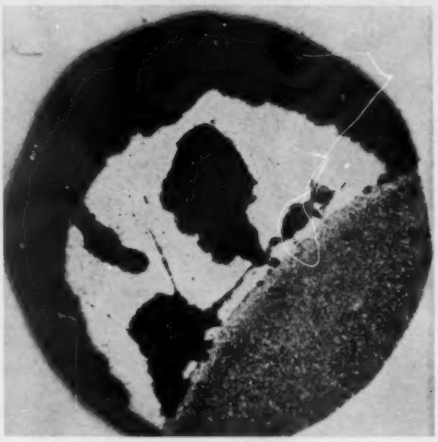
Fig. 2 shows a longitudinal section made through the wire at the original point of fracture. This is at a low magnification, and you will note right at the broken end a different shade of coloring; it is much lighter than the rest of the material.

At high magnification, the structure was found to be entirely different than that of cold drawn wire. Such a structure could have been produced by exposing the material to a temperature of 1100° to 1200° F. The wire away from the location of failure was normal for cold drawn wire.

As a result of this examination, it was evident that the wire had been accidentally heated—possibly by an oxy-acetylene torch

Godfrey Fig. 3 — Surface view of wire showing deposits of metal.

Godfrey Fig. 4 — Cross-section through wire and deposited metal on surface.



Godfrey Fig. 5 — Cross-section through wire showing martensitic structure on surface.

which was used on the job for cutting the wires. Heating of this magnitude would anneal the wire to such an extent that its tensile strength would be considerably reduced.

Another example of the effect of temperature on the properties of high strength wire is illustrated in Fig. 3. It is the surface view of a wire which had broken during the pretensioning operation. At the point of fracture and at other surface areas on the wire were what appeared to be deposits of metal.

A metallurgical examination of the metallic globules on the wire surface showed that the superimposed metal was of lower carbon content than that of the wire, as shown in Fig. 4.

The lower part of that is the wire itself. And, the globule on top, having a lot of holes, is a foreign material that had been deposited on the wire.

Directly beneath the surface deposit the structure of the wire had been changed to a martensitic structure which is very hard and brittle. A cross-section through the wire illustrating the hard martensitic structure on the wire surface is shown in Fig. 5. The martensitic structure is the white area on the surface of the wire. Martensite is formed by the heating of steel to above its critical temperature, followed by a rapid quench.

It was apparent from this examination that molten metal, possibly from an electric arc welding electrode, had been accidentally deposited on the wire surface.

The effect of high temperature on the properties of high strength steel wire, is illustrated in Table I.

The original tensile strength of the wire was 280,000 psi with an elongation of 5.2 per cent and a reduction of area of 45 per cent. We then took a piece of this wire and heated it to 1,200°F. and then allowed it to cool slowly in the air. The tensile strength decreased to 166,000 psi, the elongation was improved to 5.7 per cent, and the reduction in area was slightly reduced to 41 per cent.

A third sample was heated to 1550°F. and then water quenched. Its tensile strength was 33,000 psi. Now, this is a fictitious figure, since the sample being so brittle broke in the jaws of the testing machine. It had no elongation, and no reduction of area.

I brought some samples along, and I

Table I

EFFECT OF TEMPERATURE ON HIGH STRENGTH STEEL WIRE

CONDITION	Tensile Strength p.s.i.	Elong. in 10" %	Reduction of Area %
As Manufactured	280,000	5.2	45.0
Heated to 1200°F, Air Cooled	166,000	5.7	41.0
Heated to 1550°F, Water Quenched	33,000*	0	0

* fractured inside testing grips.

The above data illustrates to what extent certain heat treatments can effect the properties of high strength steel wire. The specimen heated to 1200°F, which is below the critical temperature, suffered a serious loss in tensile strength although it maintained its ductility.

thought maybe we could demonstrate better just what happens to this material if you do not treat it properly. This is a piece of center wire out of a strand. And, it is very high strength and elastic and real tough. Now, after heating it to above its critical temperature and then rapidly quenching it, if you bend it, it just snaps off. That is what happens.

Of course, this is a rather drastic treatment. Although you have a high quality steel you just cannot treat it badly.

As stated earlier, the number of problems of this type encountered in the field are extremely few, and it is hoped that this discussion will help to eliminate them completely.

CHAIRMAN DUBOIS:

I would like to add one thing just as a word of caution for you men who may be entertaining the thought of setting up prestressed concrete plants, and that is, (a) Do not let your welders near this high tensile steel wire or strand if possible, and, (b) choose your site carefully and away from certain chemical plants. I am sure that these steel men will give you all the advice you need on the type of chemicals that are dangerous for this very high quality strength prestressing wire.

(Editor's Note: Mr. R. O. Kasten of the Union Wire Rope Corporation was the next speaker on the panel. His discussion—"High Strength Steel Strand Crosses Lake Pontchartrain"—was printed in its entirety in Vol. I, No. 1 of the PCI Journal published in May, 1956.)

MR. SCHWAB:

To begin with, I want to state that I am a Structural Engineer, not a Metallurgist, and that I am here as a representative of the Sales Department of American Steel & Wire Division, United States Steel Corporation. In my contacts in the prestressing industry, it has been asked frequently why large variations have been reported in the modulus of elasticity of the small-diameter seven-wire strand commonly used in this country for pretensioning, and I thought it might be well to see just what is known about this quantity and its significance.

A large range of values, from 26,000,000 psi to 29,000,000 psi or even more, has been reported and the question naturally arises as to what causes this. There are a number of factors that must be considered, but first it might be well to inquire just what is meant by the term, "modulus of elasticity."

An extensive, although not exhaustive, search through technical literature has failed to produce a generally applicable definition. A comprehensive statement is, "Modulus of Elasticity is a numerical relationship between stress and strain." As far as steel is concerned, the modulus of elasticity is generally considered to be the constant ratio of stress to strain below the proportional limit. Obviously, this definition has no significance when applied to materials lacking a proportional limit or to any material for stresses beyond the proportional limit.

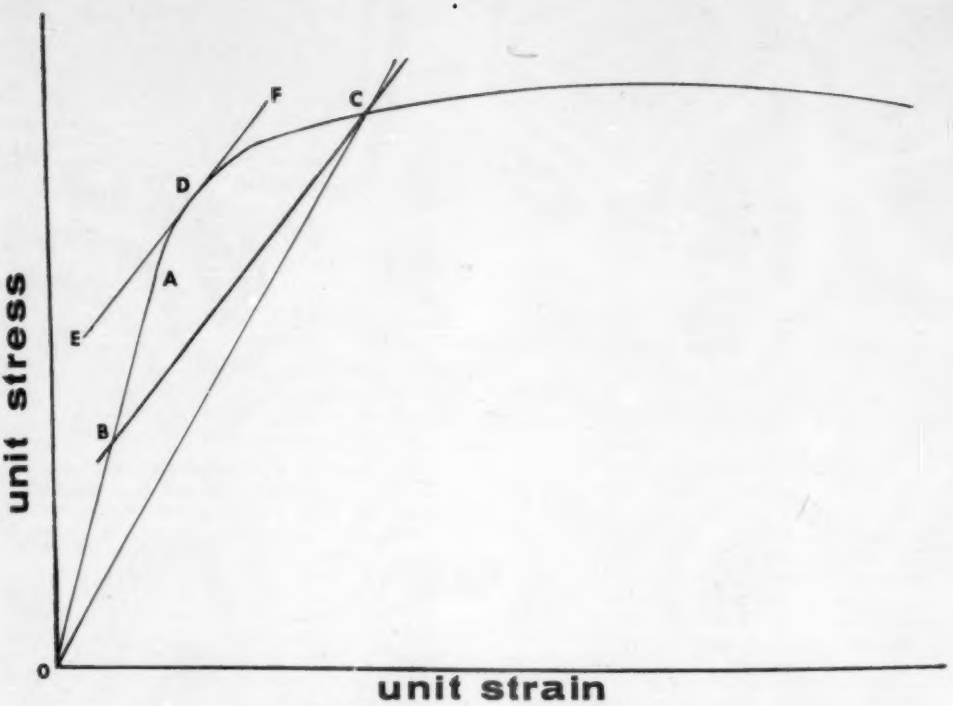
With relation to stranded wire, the use of "modulus of elasticity" is somewhat inaccurate. Actually it is a property of a material and its numerical value depends on the type of stress involved. However, usage of the term is so well established in the prestressed concrete field that it will be employed here in lieu of some more exactly descriptive phrase. Since strand is used only in tension, it must be understood that consideration is being given only to the relationship between tensile stress and strain and that the moduli of elasticity discussed are tensile moduli.

The use of the term "modulus of elasticity" is frequently modified or restricted. Some of these modifications can be understood by reference to Figure 1. This is an idealized stress-strain curve similar to that for steels used to prestress concrete. The proportional limit is indicated at A, and the stress-strain relationship is a straight line below that point. The slope of that straight line is what is usually taken as the modulus of elasticity; it is sometimes called the "initial," "tangent," or even "initial tangent" modulus of elasticity. Unless otherwise specified, modulus of elasticity is usually understood as initial modulus of elasticity.

For a material without a proportional limit and hence having no initial straight-line portion of the stress-strain curve, the initial modulus of elasticity is taken as the slope of a line tangent to the curve at the point of zero stress and strain.

Any line, such as B-C, cutting a curve in two points, is a secant. If the lower point B is taken at the origin O, the slope of the line O-C is called the "secant modulus of elasticity." The line E-F is tangent to the curve at point D and its slope, which is also the slope of the curve, may be called the "instantaneous modulus of elasticity." In any case, the modulus of elasticity is a slope and is equal to a stress divided by a strain. The definition previously given can be expanded to read, "Modulus of elasticity is a numerical relationship between stress and strain and for any change in stress is equal to that change divided by the accompanying change in strain."

A determination of modulus of elasticity can be made in two ways. The elastic curve can be obtained (either by making isolated simultaneous readings of strain and stress, plotting them, and drawing a curve or automatically with electric strain and load recording instruments) and the points between which the modulus is desired located by eye. A second method is to make stress and strain readings at only the two points and to calculate the slope directly. Either



Schwab Fig. 1 — Idealized stress-strain curve.

method is subject to mechanical and visual errors, although error due to plotting a curve through a number of points is not involved in the latter method. The two-point method could, under certain conditions, yield erroneous results because it does not take into account the statistical scatter of isolated readings.

When measuring modulus of elasticity from zero stress, any initial curvature of the specimen may materially reduce the apparent value as a portion of the applied load will merely straighten the piece without elongating it axially. For stranded wires a complication that does not occur with solid wires must be considered. As the strand is loaded, the outer helical wires try to straighten and the strand becomes more compact. This effect is known as structural set, and the resulting elongation as structural stretch.

In making a stress-strain curve for a wire product, a small initial tension is sometimes placed on the specimen to straighten it, and thereby eliminate, or at any rate materially reduce, the effect of curvature. The load is released and the stress-strain loading recommenced from zero. Now as a strand is loaded some of the structural stretch is removed, the amount depending on the magnitude of the load and the length of time for which it is applied. The effect is to raise the apparent modulus of elasticity, and the initial tension may affect appreciably the calculated value. It may be necessary to distinguish between a "first-run modulus of elasticity" obtained with a strand from which little or no structural stretch has been removed and a "second-run modulus of elasticity" for a strand from which a large portion of the structural stretch has been removed.

A factor that seems to have a marked influence is the gauge length in which elongations are measured. Those commonly used commercially are 10″, 20″, 24″, 50″, and 100″, while a length of 100 feet is used in the Research Department of American Steel & Wire. Without attempting to say that one is better than another, it appears that longer gauge lengths yield more consistent results. Certainly, if loads and strains can be measured with the same precision, errors should be less significant. While I cannot speak with authority for the Research Department, I have been given to understand that the 100-foot bed is considered to be as accurate as any method now known.

The effect of curvature of the specimen is probably less pronounced. When extensometers are used, the position of the extensometer (whether on the outside or inside of the curve, or on one of the uncurved sides) can affect the strain readings. In the 100-foot bed of AS&W, the strains are measured only at one end of the strand and hence this problem does not occur. Another reason why results obtained with the longer gauge lengths are more con-

sistent is probably found in local differences in the strand. A test on a 100-foot long specimen is in effect an averaging of 100 tests made on one-foot long specimens, and hence should eliminate much of the effect of such differences. It might also be noted that the 100-foot gauge length most nearly approaches the situation in an actual pretensioning operation. Unfortunately, however, this procedure is obviously not suited for production control methods as it takes too much time and material. It is just as obvious, however, that there is a need for a better, or at any rate more standardized, method of testing than is now employed commercially.

Another interesting demonstration of variations in results from different methods of testing was obtained from two sets of samples of the same strand. Tests were made at the New Haven Mill of AS&W with a 10" gauge length Microformer automatic strain recording mechanism, at the Research Department using a sliding vernier to read strains on the 100-foot stress bed, and by another manufacturer using a 20" gauge length extensometer with Ames dials. Moduli of elasticity obtained were 25,300,000 psi, 26,480,000 psi, and 27,100,000 psi for one set, and 26,400,000 psi, 27,300,000 psi and 28,100,000 psi for the other set. These, of course, are only isolated tests, but they do indicate the differences that are encountered.

Perhaps the principal source of difficulty is the fact that there is no standard method for testing prestressing strand, and for determining its modulus of elasticity. Each producer and many testing laboratories have developed their own procedures and, because of the things indicated previously, differing results are obtained. There are also other items, such as the way that the strand is gripped in the extensometer, that can have an appreciable effect on the results. It may be that with the increasing importance of prestressed concrete, the American Society for Testing Materials will produce a standard method of determination, but until something like that is done the variations that have been reported can be expected to continue.

In some cases construction specifications have been written requiring a certain minimum or average value of the modulus of elasticity. This is really meaningless unless the testing procedure is also specified. In this respect, a somewhat similar case is the determination of elongation at rupture in a specimen of structural steel. This means nothing unless the gauge length in which it is to be calculated is stated. In the case of average values, an average cannot be obtained until the last sample is tested. This may not be practical in a large job where some of the strand is incorporated in the structure before another portion is manufactured.

Lest the large range of values for modulus of elasticity seem alarming, it should be remembered that the modulus for other materials may be equally variable. It is quite common to think of that for structural and reinforcing steel as 30,000,000 psi, but a considerable range of values can be obtained. Methods of obtaining loads, areas, and strains, the condition of the sample, and deviation from true axial loading can have quite an effect on the results. Reliable authorities cite a range of 28,000,000 psi to 31,000,000 psi, more or less.

In reinforced concrete design, it is of fundamental importance to know the moduli of elasticity of both reinforcing steel and concrete, or at any rate their ratio "n". However, "n" is assumed by the crudest sort of assumption, and without alarming anyone. No engineer would ever think of specifying the modulus of elasticity of the reinforcing steel and obviously could not specify that of the concrete.

As far as prestressed concrete design is concerned, the modulus of elasticity is employed in three ways. If transformed areas are used, they must be calculated using the modular ratio "n" but this is fixed in such an arbitrary manner that a considerable variation in the modulus of elasticity of the steel is insignificant. When a prestressed concrete beam is loaded, the change in steel stress due to loading is also a function of "n", but this change is small, unimportant, and generally neglected. When stress losses due to creep, compression, and shrinkage of concrete are computed, the loss of steel stress again depends on the modular ratio. Thus loss is also generally fixed rather than arbitrary; so again the modulus of elasticity does not have too much importance.

With regard to construction, when stress is to be determined by measured strains, it is of primary importance to know how much to stretch the strands in order to apply a specified tension. Practice today is divided between this method and the use of calibrated gauges or dynamometers. In the latter case, extensions are merely useful as a check on jacking operations and for this purpose some approximate value, say 27,000,000 psi, may be used satisfactorily.

Even if elongations are used to establish stress, it should be remembered that this is the initial stress. The final stress which, of course, determines the behavior of the beam in the early load stages, is the initial stress less whatever losses may occur due to stress-relaxation, shrinkage, creep, etc., but as has been mentioned, these are fixed arbitrarily or estimated on an empirical basis and the residual stress is not known with any accuracy. Ultimate capacity of the beam is not affected materially, if at all, by the prestress. These things indicate that great concern about the exact modulus of elasticity is unjustified.

In this regard, it should be pointed out that load, rather than unit stress is observed when making a tension test. To obtain the true stress, the load must be divided by the actual area. However, in construction work,

the estimated elongation is in most cases obtained by dividing the applied load by the theoretical cross-sectional area. This may also result in an error of several per cent, and is another indication that exact values of modulus of elasticity are not important. Actually, it would be more helpful to have the *strain* corresponding to the initial *force* in the strand rather than the modulus of elasticity.

In conclusion, it seems that at present, any concern about exact moduli of elasticity of pretensioning strand is unjustified, both because of uncertainties in the method of determination and relative unimportance in the design of prestressed concrete. For all practical purposes, satisfactory results can be obtained by use of some reasonable approximate value. A minimum value would be useful in guarding against loose stranding, but because of lack of a testing standard and insufficient experimental data, it seems unwise to select that minimum at the present time. The large range in the values that are currently being circulated should not be considered as reflecting irregularities in the material but rather uncertainties in the testing procedure. Our engineers are continually attempting to obtain a better method with more consistent results and, of course, all of our efforts will always be to manufacture the best possible product.

MR. LEVINE:

I am going to discuss a number of systems that have been employed in the stressing process in the last few years, and point out the highlights of each particular system.

First is a double T bed with a single jack, where the jack pushes a crosshead, pulls the template forward with a system for taking up the nuts during the detensioning operation. The same jack can be used for, say, a pile section or an I-beam with a single point jack. Here, we use the same tensioning operation and the same detensioning operation.

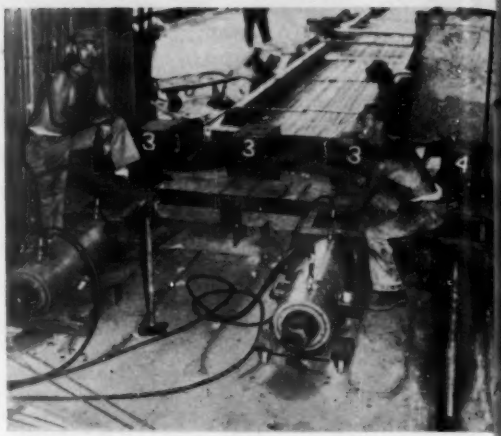
In the two point jacking system the crosshead is in the front, and the template is also mounted in front. The crosshead is moved forward until you attain the pretensioning load, and then the jacks can be transferred to another bed.

In the four point jacking system the lower jacks are not advisable. The side members acting as struts are placed at a diagonal to the vertical line of the two jacks.

The template here can be removed and either welded or bolted on very simply using the same crosshead without a change, so that you might cast an I-section or a channel or any other shape you might want.

The jacks with the lighter load would tend to run out faster than the jacks with the heavier load, distorting, possibly, your center screws. So, a system of controls has to be installed by which these jacks could run out equally and record the pressures independently.

Now, under the four point jacking system,



Levine Fig. 1 — Example of typical four-point jacking system.

as shown in Fig. 1, we set up two gauges on each jack so that if there is any differential loading, we have four points on which we can take a recording of the pressure.

There is one additional advantage, and that is for most cases, you do not have to transfer or move your jacks up and down, except for extreme cases where you may have all your strands bunched together beyond the working pressure of that jack. This system permits you to pick up the jacks and transfer it to a double T, fitted into a slot and commence operation there.

Briefly, I would also like to discuss how to analyze your jacking requirements in terms of your present and future operations. Your operating pressure should be about 6000 psi. I have had beds which started at 6,000 and went up to 7,000 psi and have been highly successful, but you should have some reserve. I would say that a great number of beds are now being set up with tensioning loads of about 800 kips and in a number of installations they are using a four point jacking system of 150 tons each.

Now, you have one other problem. The frictional component in the jack can very often be a source of discrepancy, the difference between your elongation measurement and your hydraulic pressure. The frictional component can be computed on some sort of a testing machine. And, I have found that on jacks, having six microfinish and 12 or 15, there is considerable variation at a 1000 psi—anywhere from 10 to 24 per cent on some models of jacks. So that if you took a 300-ton jack and operated it, say, for a 50-ton job, that error will be a lot greater than if you used a 300-ton jack for a 200-ton job.

QUESTION:

In view of the coefficient of friction in the jack, would it be better to use the elongation rather than a very sensitive pressure gauge to determine the elongation? In

other words, which is the best system to use?

MR. SCHWAB:

I cannot say which would be best. That was one of the points that I wanted to make, that if we knew what the elongation was that corresponded to a particular situation, that would probably be more reliable. But, we have no method of determining the elongation in such a way that will correspond to the situation that occurs on the prestressing bed. If we determine the elongation with a commercial testing apparatus having a fixed gauge length the question of curvature of the strand comes into the picture. The curvature of the strand is probably insignificant for a casting bed three or four hundred feet long. So, unless you have some way of correlating your test data with the situation that occurs in the prestressing plant you cannot say that it is more reliable than the jack. Under any circumstance, the error of four per cent that Mr. Levine mentioned would probably be no less than the difference that is to be anticipated if you were to use some fixed value of elongation that was determined by some commercial testing method.

You must remember that steel is not a homogenous material. Even though the manufacture of strand is more carefully controlled than any other steel, there will still be variations that will occur. And, those variations will be, very likely, in the order Mr. Levine mentioned on the jacks, and you cannot say arbitrarily that one is more accurate than the other.

QUESTION:

Can we consider that those differences would be minor when we are using arbitrarily a 15 to 18 per cent stress loss? Do you think it will be as a practical matter to measure elongation?

MR. SCHWAB:

As a practical matter, I think it would be as well to use either method. Now, that is a hedging answer, but that is the exact situation. As far as the effect on the end product is concerned, either method will give satisfactory results.

QUESTION:

Is there a difference between Type I and Type III cements with respect to shrinkage and plastic flow?

MR. GRUENWALD:

There is a slight difference. I have some figures here on some tests, and the difference is not very great—it is in the third significant figure. We ran some tests on Type I cement having a two inch slump. Regardless of what we used as the cement content, from three and a half to seven and a half bags per cubic yard, the average shrinkage was one inch per hundred feet. And, on a seven and a half bag it ran about 0.67 inch per hundred feet. On the same type of mix with a Type II cement, it was 0.66, and on the Type III cement it was 0.68. So, there isn't much difference.

On the plastic flow of concrete, I really do not have any information. That depends much more upon the age at which you apply the load. I believe if the load is applied on concretes of about the same strength, you will get about the same plastic flow or creep.

QUESTION:

What can you do with your warm concrete in the summer when your outside temperature is 90 or 100 degrees?

MR. GUSTAFERRO:

Mr. Gruenwald said to make it cool. I think that is the answer. Now how do you make it cool?

The easiest way is by cooling the mixing water. The mixing water, of course, has the highest specific heat, and you can cool the mixing water a lot faster than you can the aggregates. The other thing that you can do, at not too great a cost, is to cool the aggregates by sprinkling the stockpiles with cold water or even ice chips, making sure, of course, that the ice is fully melted before the concrete is placed. That will be effective in reducing the temperature considerably.

I do not think there is any need ever to have concrete at 90 degrees.

QUESTION:

When you speak of keeping it cool, you mean for just four hours and then you intend to increase the heat for curing purposes?

MR. GUSTAFERRO:

I mean keeping it cool only during the placing. What happens to the temperature after it has been placed is not near as significant as the original temperature of the concrete. I do not mean to keep the concrete at 40 degrees or anything like that for any length of time.

QUESTION:

You said a little while ago that 70 or 75 degrees was the best to keep it at for about four hours.

MR. GUSTAFERRO:

No.

QUESTION:

Is that four hours pretty well accepted as the time before you should increase your curing temperature?

MR. GUSTAFERRO:

Well, I think you would not even have to wait the full four hours. I think a couple of hours would probably be sufficient.

QUESTION:

In casting slabs which have relatively thin sections (two inches) the heat of the form will increase the temperature of the concrete from 75 to 80 degrees at placement to as high as 110 or 115 ten minutes later. Do you think this will damage the strength of the concrete?

MR. GUSTAFERRO:

In that case I would suggest, if it is at all practical, to attempt to cool the forms. If you are using steel forms, you could cool them by wetting the forms. The pans are sometimes very hot, and in such cases the concrete is actually shown to be of insufficient strength right at the form. I do not think hot forms are ever good for the concrete.

Second Annual Prestressed Concrete Institute Convention

PANEL DISCUSSION

Specifications

CHAIRMAN

R. W. Kluge, Civil Engineering Dept., University of Florida

PANEL MEMBERS

Thor Germundsson, Portland Cement Association Chicago, Ill.

Carl E. Ekberg, Engineering Dept., Lehigh University, Bethlehem, Pa.

H. J. Godfrey, John A. Roebling's Sons Corp., Trenton, N. J.

Dr. T. Y. Lin, University of California, Berkeley, Cal.

William E. Dean, Bridge Engineer, State Road Department, Tallahassee, Fla.

MR. GODFREY:

Specifications are a general subject and discussing figures sometimes gets to be rather boring.

The properties of small diameter prestressed concrete strands are not specifically described in the preliminary report of the Joint Committee except to state that "the strand property should be substantially the same as that of high tensile wire."

We would like to submit the following proposed specification on uncoated seven-wire strand for your consideration.

"The manufacture of steel wire and strands shall be conducted in accordance with the best current practice of high-quality drawn wire. All strands shall be of the seven-wire type, having one center wire and six outside wires. The center wire shall not be longer than the outside wires to guarantee that each of the outside wires shall bear on the inside wire, thus gripping it. During fabrication, wires may be spliced in accordance with the best welding procedure, but no splice shall be made within 150 feet within a splice of wire.

"In the finished strand the six outer wires shall have a uniform pitch of not less than 12 and not more than 16 times the nominal diameter of the strand.

"All strands shall be stress-relieved as a unit after the wires have been formed into a strand.

"The wires shall be of carbon steel having an analysis in the following range: carbon, 0.70 to 0.85 per cent; manganese, 0.40 to 0.90 per cent; phosphorus, 0.045 per cent maximum; sulphur, 0.050 per cent maximum.

"Strand properties shall conform to the following table: A strand diameter of one-quarter inch with an approximate area of 0.356 square inches would have an ultimate minimum strength of 9,000 pounds. A strand diameter of five-sixteenths inch with an approximate area of .0578 square inches would have an ultimate minimum strength of 14,500 pounds. A strand diameter of three-eighths inch with an approximate area of .0799 square inches would have an ultimate minimum strength of 20,000 pounds. A strand diameter of seven-sixteenths inch with an approximate area of .1089 square inches would have an ultimate minimum strength of 27,000 pounds.

"The minimum yield strength, measured by the 0.2 per cent offset method shall be not less than 85 per cent of specified minimum ultimate strength.

"The elongation of the strand shall be not less than four per cent in 24 inches.

"The elongation shall be determined as the per cent increase in separation between the jaws of the testing machine from the position after application of the initial load to the position at the initial failure in the test specimen. The separation of the jaws of the testing machine shall be approximately two feet when under an initial load equal to 10 per cent of the required minimum breaking strength of the strand. The elongation values shall be recorded for all specimens even though the fracture takes place in the jaws of the testing machine. In the latter instance, however, when the material fails to meet the elongation requirement, two re-tests shall be allowed.

"All tension and elongation tests shall be made on lengths of the strand which do not contain wire splices.

"On request, the manufacturer shall certify that a sample of the finished stress-relieved strand has been taken from each 100,000 feet of strand and that these samples have been tested and have met the requirements of these specifications.

"At the customer's request, and without charge, the manufacturer shall furnish a typical load-elongation curve for the size of strand being furnished.

If the customer requires tests or curves on the specific lot of strand being shipped to him, they shall be made at his expense.

"The proposed specifications will guarantee the properties of the strand within practical limits and assure satisfactory performance in all respects."

MR. GERMUNDSSON:

I would like to give you a brief resume' of what we have been doing in the ACI-ASCE Joint Committee 323 on specifications for prestressed concrete. I will just give you an introduction today and a more thorough discussion will be presented on Friday.

This Committee was actually organized, I think somewhere around 1944. At that time there was only one place in this country where they were making prestressed concrete and that was the Borsair Tank Company in St. Louis.

Now, the Committee did not do much of anything for a long, long time. Their idea was to study, investigate and guide, but nothing much came of it until 1952 at the Centennial of Engineering in Chicago, when we had a big meeting and we reorganized the whole Committee. As a little bit of an alibi, I must say this: At the meeting and prior to the meeting, there had been a great deal of controversy as to what the Committee should do. We divided ourselves into two groups for and against writing a code. Those who were against the code said that a code might have a finality that would hamper future developments. Those in favor of a code said that we needed it—and we needed it badly. We needed it fast and we called attention to the fact that the ACI Building Code, 318, started way back in 1909 or 1910 and it had been revised every two years ever since; and there was no reason on earth why the Prestressed Concrete Code could not go through the same kind of development.

However, the group that was against writing a code won out and another avenue of approach was adopted, namely, that we should draw in all the experts in prestressed concrete all over the country. We should divide them up in task-committees and each task-committee was to take a specific part of the code and work out a manual type of a contribution. So we added about 90 members and divided them up. However, that was extremely unwieldy. People just do not work that way. It was non-uniform in the respect that some task-committees came across in fine shape. Others did not do anything. Those who came across with good suggestions were hurt because the rest of them could not catch up, hence, nothing happened. In August last year the Committee was reorganized, but in the meantime, Eric Erickson of the Bureau of Public Roads had come out with a *Criteria* which then took the place of a design and materials specification. Mr. Erickson is Code Chairman of the Committee 323.

Now, we have everything in from the task-committees and that was a lot of work. Also, we have gone all over the various types of foreign specifications. We have extracted them and worked out a big file. We then cut that down into a report which I hold in my hand, which has something like 80 pages. It is a preliminary report which is not to be used in any other way than as a basis for discussion and comments. We presented that at the ACI Convention in Philadelphia in February of this year.

We had a long four-hour meeting with all kinds of excellent contributions. We have received a lot of comments by mail. The time to revise this document has not come yet. I would invite everyone in this region, particularly, where you have so much experience with prestressed concrete, to feel free to comment on these specifications. Our future program may look somewhat slow to you, but that is the way it has to be. In order to get into the ACI, the standards have to be submitted to the Standards Committee not later than September 1st. That leaves June, July and August, and we can not possibly get this revised in three months and have it voted on and make the corrections necessary. So we will have to shoot at a year from September and then shortly after that it will probably be printed in the ACI as a progress report, which may or may not carry enough weight with various municipalities to be considered as an official specification.

Now, I do not think you have had too much trouble in getting your prestressed concrete adopted in the State of Florida, but there are all degrees of acceptance by municipal authorities. One city has a particularly cantankerous engineer. He will not let one piece of prestressed concrete into the city until he has an official specification. In other cities, they do not even question it. Then, we have all degrees in between these two extremes. In at least three cities, that I know of, we have been able to put into the City Code reference to prestressed concrete so that it will be accepted on two bases: Either that it will be accepted on the basis of acceptance tests or that the *Criteria* will be used as an official specification when available. We have had some very good luck with a few cities that have asked that we work with them.

A remark now about specifications. This has to do with fire resistance. Those of us with the other panel this morning heard Mr. Godfrey mention some tests that his com-

pany has made where they found that wire with a 280,000 psi strength would drop down to 166,000 psi at a temperature of 1200°F. Actually, that is a little better than I had thought. Now, the way the fire resistant tests are set up, we have to have a certain amount of strength left before the specimen collapses under the superimposed double live-load and the main difference between the prestressed concrete and the ordinary reinforced concrete is that the prestressing wires will lose their strength a little bit sooner than the reinforcing bars. An allowance will be made by more concrete cover to keep the temperature of the steel from going up as fast as it would if it were closer to the surface.

Now, the Portland Cement Association does not like to have this discussed too much because our plans are not in final shape. However, I can say that there has been an extremely active procedure in the Portland Cement Association in the last nine or ten months about the fire resistance of prestressed concrete. You will have to realize that we have to have a building built. Within that building we have to have furnaces. Just take the furnace alone. We would have to discuss whether to have a rolling yellow flame or downdraft with a blue flame. We have to discuss whether the loading that has to be put on has to be put on one way or another. Altogether, nobody has too much experience with ovens made for testing specimens for fire resistance; but that has all been gone into very thoroughly and plans have been made and we hope we can raise the money to get going very soon. So, in the meantime, if you run into fire resistance problems, hold off until we can really get going.

T. Y. LIN:

I presume you all know I am allowed ten minutes to discuss this extremely intricate problem of the law of stresses. Of course, I can always refer you to my book. Actually, I think the matter of stresses has been a rather confusing problem. As engineers, we tend to scare people off in talking about stresses. When we talk about tension, shear and bond, they can not talk very much against us. As professors, we can give students exams on allowable stresses. As building officials, we can always check the stresses in the designs, if we cannot check anything else. I think the word "stresses" is actually one of the worst misnomers we have in the history of engineering. Very often stresses we talk about are not real stresses in construction. Very often they are the real stresses, but they have no real meaning. A structure can be terribly overstressed and serve very well. Another can be understressed and fail. Take a steel beam and a connection, using a standard angle. That connection is always overstressed. We just do not compute it. Since it is never computed, no building official ever questioned it and no professor ever teaches it; and it goes into the building and never gets us into any trouble.

I think the two basic requirements or criteria for design are these: First, the structure must behave properly under the service conditions. When you design a structure, it is implicit that it should not crack too much, it should not vibrate too much and it should not fail under fatigue loads. Behavior under service conditions is one criterion.

A second criterion is strength. We know that any highway bridge will get a 100-ton truck and the bridge would still stand. We have a factor of safety and excessive strength. We do want to provide excessive strength. Now, if we design our structures to satisfy these two basic requirements, I would say the structural requirements are met. That is, they will behave properly under the normal service conditions with or without overload. In case of overload, they can still take it. We can design prestressed concrete that will not be overstressed, as far as stresses are concerned; but it may deflect too much. We have seen the troubles of prestressed concrete as well as other designs which can meet all the requirements of the stresses, but will not satisfy other requirements. We can also design a structure that will behave quite well under normal conditions, but if you overload it just a tiny bit, it will fail. We know there are certain designs in prestressed concrete which would seem very good at the start, but if you overload them just a little bit, they will fail. There are also very strong designs in prestressed concrete which are barely satisfactory under service conditions, but if you overload them four or five times, they will not fail.

We tested one slab, a prestressed slab, 15 foot square, supported at four corners. We computed the stresses by the elastic theory. According to the elastic theory, if we put one-pound load on that slab, five inches thick, it would go into tension. Some building official would throw up his hands. We tested it at 120 psf before it began to crack. We loaded it to 240 psf before it eventually failed. If you apply the rule of no tension, the allowable load is one psf, while we can certainly design it for 80 psf or more.

I think it is most difficult to design prestressed concrete by fixed allowable stresses. Design your structures for the basic requirements of satisfactory behavior and sufficient strength.

MR. DEAN:

Any design specifications will necessarily have to be based on rational design concepts, which are themselves based on the laws of mechanics of materials. By composite construction, what I will be referring to is the type of construction in which one part is manufactured first and usually erected and supports the load of a subsequent part; and the two parts, acting together, resist the moments and shears induced by the superimposed load. That

assumption of interaction can only be realized by the development of the shearing forces, acting between the two elements.

Now, most of us, at least those of my age in structural practice. were introduced to composite construction very largely through the patents and applications of the Poring Company. These are the people who first put out, patented and advertised the connection between an I-beam and a concrete slab. They put out a wonderful connection which would resist both horizontal and vertical forces. For shear effectiveness, I do not think it has ever been surpassed by anything that has been subsequently developed.

The interest in composite construction has been the subject of considerable research, a good bit done at the University of Illinois, with contributions from various sources all over the country. Investigations have been made of many bridges with I-beam structures. It has been found that through the years on the highways, practically all of the concrete slab bridges which were constructed on wide-flange beam sections without any mechanical device to develop the shear between the concrete and the steel, have been acting as a composite structure. The natural bond, the adherence of the concrete to the steel, has been such that in spite of the density of the traffic and frequent overloads to which most highway bridges are subjected, in most cases composite action has prevailed through the years. Those on which composite action has been lost have been principally structures of short span narrow flanged standard American beams.

Most of the specifications that cover composite construction seem to put great emphasis on the proper development of the horizontal shear between the original element and the superimposed element. It is a matter of very elementary mechanics of materials that the force between these elements is practically always an inclining force. In fact, where actual stress is small or non-existent, the forces and combined forces of 45 degrees have equal horizontal and vertical components. That being the case, I have often wondered why emphasis is placed on horizontal shear and very little mention is made of the vertical uplift.

All precast concrete construction is particularly adaptable to composite type of construction. Thicker slabs can be poured on originally thinner slabs. Lintels can be filled up and the full section considered as carrying the live loads or superimposed loads.

With this idea that the horizontal component of the force between the elements was overemphasized, it seemed to me that we had been going to an awful lot of trouble in our concrete members by constructing these troublesome shear keys on the top of a beam or slab. They are nuisances in construction. They foul up quite frequently the bottom plane of reinforcing steel. If they can be gotten rid of or minimized, certainly we will have simplified one of the construction operations. That being the case, it seems to me that if we extended enough steel from the original beam into the superimposed slab to resist this tendency to uplift, the natural bond between the beam and slab would certainly be sufficient so that horizontal sliding could never occur. We based some of our designs on this concept and it was not received with complete enthusiasm by the Bureau of Public Roads which is the organization that we have to work with in all of our work.

So, quite logically, a test was set up over in the Tampa Bay area. We constructed one large beam having smooth trowelled finish on top with enough stirrups extending through to develop the vertical force. We poured the slab on this beam. Then we loaded it by putting on a very heavy overload in shear—about four times the normal load. Then we moved the load application to the center of the beam and it took over five times the normal load to bring about the destruction of the beam. At no point in these tests was there the slightest indication or any sign of distress whatever at the junction between the beam and the slab.

One of the matters in the current progress report of Committee 323 which bothered me a good deal was a stipulation that under no circumstances could bond be depended upon for development of the shearing force between elements in composite construction unless the initial member had, I believe, 70 per cent of the periphery wrapped up by the superimposed concrete. That seems most unrealistic to me. There is a powerful adherence both horizontally and vertically of concrete to concrete. In any structure subjected to dynamic loading, I would certainly hesitate to depend upon that bond entirely. In a building, which is not subjected to heavy overloads at frequent intervals and is not subjected to impacts, I see no reason why the assumption of composite action could not be applied to just simple concrete on concrete with possibly only a nominal connection for the holddown. My own idea of this composite construction is to hold the two elements together vertically. Then, with just a normal amount of roughening of the surfaces, I firmly believe that the resistance to horizontal sliding to be safely taken is about the value that we would apply on bond in reinforcing steel.

DOCTOR EKBERG:

My subject is to discuss the fatigue properties of prestressed concrete. In general, it can be said that there are no direct recommendations concerning fatigue in specifications on prestressed concrete nor is there much published information about the fatigue properties of the materials involved.

It is an established fact that the lower the range of stress under loading, the higher the ultimate dynamic strength. This will hold true for prestressing steel as well as for concrete. A few tests on the fatigue properties of small pretensioned beams have been made which bring out the fact that there is a distinct advantage in going to as high an initial steel prestress as possible for a given design. For example, the initial steel prestress of one beam was 140,000 psi and that of a second beam was made 100,000 psi. The dynamic ultimate moment came out 20 per cent higher for the beam with the higher prestress. I expect to go into more detail on this point in my talk on Friday.

I would like now to take a quick look at the Bureau of Public Road's *Criteria* to see if it seems adequate to cover fatigue loading of prestressed concrete. One condition in the code which applies is that giving the stress of steel under dead and live-loads as six-tenths of ultimate strength or eight-tenths of the yield strength, whichever is less. A second condition sets the ultimate computing moment at either the moment due to dead-load plus live-load plus impact or about the moment due to twice dead-load plus live-load plus impact, whichever of the two governs.

In view of what we have mentioned concerning the importance of stress range with repetitive loading, it appears first of all that the requirements of the code may not be strict enough in that they do not stress a limitation on the range of stress due to live-loads. The range of stress due to live-loads moving on and off a pretensioned span will in general be greater for the shorter spans than for the longer spans. It might seem advisable to inject an additional requirement that the sum of the steel stress be for live-load plus, say, five times the increase in steel stress due to live-loads should be made less than the dynamic ultimate strength. The factor five is believed to be sufficient for safe practice. This requirement would lend itself well to fatigue because it allows us to go as far as possible with our steel stresses, regardless of the span. It correctly yields a relatively low required steel stress for the more severe dynamic stress for a short span and a higher steel stress initially for the less severe stress for the long span. The Bureau of Public Roads' provision that the computed ultimate moment be greater than the moment due to twice dead-load plus live-load plus impact might be thought of as a provision against fatigue. However, it is extremely conservative for long spans because of the large dead-load. The dead-load of a bridge is normally calculated quite accurately and therefore the factor of two seems not to be justified.

The other condition, namely, that the computed ultimate moment be greater than the moment due to dead-load plus three times live-load plus impact fits well to the requirements of the fatigue.

Now, in conclusion, it is recommended first that consideration be given to exclude the requirement that the computed ultimate moment be greater than twice dead-load plus live-load plus impact for large span pretension work. Second, that consideration be given to the change in steel stress due to live-load. Third, that more research work be carried out, particularly on the dynamic stress of prestressing tendons.

QUESTION:

I would like to ask a question on the welding of the strands. This morning a gentleman talked about welding the drawn strand after it was drawn. I am just wondering about change in the metallurgical structure. There was a comment about making a test at places other than where the strand had been welded. I would like a little more information on those points.

MR. GODFREY:

What I said this morning was that you should not apply any heat to a high carbon steel wire, whether wire or strand. You will spoil its properties. Therefore, you just should not try to weld that strand to something. You are just going to ruin it.

As far as welding during the manufacturing process, we do that on a single wire in the strand and that weld occurs about every 10,000 feet in any given wire. So there are not many welds. Right at the weld you have a wire that is weaker than elsewhere. For that reason, as I will show on Friday, the tensile strength of the strand will be below its normal strength, because when you test a strand having a weld in it—when I speak of a weld, it is just one of wires in a strand that has been welded—the wire will break at the weld. At that point, of course, you have reached your maximum load and it does not measure the true strength of the strand. That is why we recommend that any specification tests be made on a strand that does not contain a weld. In your structure there are so few welds, if you consider one weld to every 10,000 feet in any one wire, that you can just forget about them in the design. If you are designing that close, you are just doing too much calculation.

QUESTION:

What I was concerned about was that in a seven-wire strand if one wire cracks, that puts a shock load on the other six.

MR. GODFREY:

There is no shock effect. I mean it would not affect the properties of the other wires at all. If the welds are made properly, you should not get any weld failures in pretensioning. I will admit, there have to be some, but, normally, I would say that if you are testing a strand that has one wire with a weld in it, you should develop at

least 80 per cent of its catalog strength before you get that weld to fail. Since you tension it to 70 per cent, you should not get any failures. Of course, a weld is something that is difficult to make and you do not always get 100 per cent quality. However, generally, a strand containing one weld will develop at least 80 per cent of its catalog strength. Do not worry about the shock load on the rest of the wires. That wire is strong and that wire is tough. It will not affect the properties of the other wires a bit.

PROFESSOR LIN:

May I add a little to this? The breakage of a wire is usually not sudden enough as to put a shock load on the others. If one of seven breaks, the others would be stressed higher by about 16%. Most of the time the six wires will be able to take that increase, and only rarely do they break all the way through.

MR. KLUGE:

I would like to ask Mr. Godfrey if it is possible to heat-treat welds in such a way so as to overcome the effect of the weld?

MR. GODFREY:

Well, there has been a lot of work done on heat treatment of welds to try to improve the properties, but you always have that heat-affected zone. I mean there is always a section on both sides of the weld. During heating, it will come up just below the critical temperature. You cannot do anything about it. When you make a weld, you have to admit that you have a weak point there and put up with it. Of course, if you want to omit the weld, it is going to cost you money because it is more expensive to make a strand without a weld in it. From an economic point of view and a practical point of view, we feel that the welds are so few and the cost difference would be so great, that we feel the most economical strand is one with welded wires.

QUESTION:

How soon would the imperfection in a weld show up after you put a strand under tension? Would an imperfection in a weld show up as soon as you get your maximum tension on the cable or would it be a delayed thing that could run into 24 hours after the concrete was poured?

MR. GODFREY:

No, I think if it is going to break, it will break during pretensioning right away. If it does not break then, you will have no trouble.

QUESTION:

In other words, it would be a visible thing?

MR. GODFREY:

Yes, it would be something you could see or hear.

QUESTION:

And if a weld were to break –

MR. GODFREY:

If it is going to break, it will break right away when you apply the initial load.

MR. GODFREY:

I have some information on fatigue that I should like to mention. The question of fatigue in wires has been brought up and, of course, in prestressed concrete, the wire is under pulsating pretension stresses. To be honest with you, there has been very little work done on pulsating tension stress.

However, I have a German magazine, Draht, for the month of April. There is a very good review by Hempel of all the fatigue tests and, apparently, they have made a much broader study in Germany than anywhere else. He reports the results of various tests, including some that were made in this country. They indicate that as far as wire itself is concerned it is quite adequate for any stresses that might be put on it in a prestressed concrete beam. To illustrate that, I will just give you some test results.

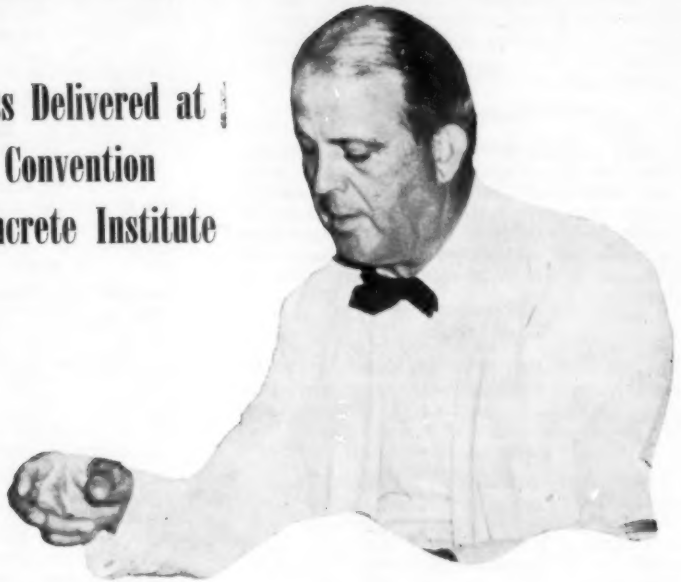
At a mean stress of 60 per cent of the tensile strength, they were able to pulsate plus or minus 37,000 psi without failure. That is quite a range of stress at a 60 per cent average.

Other results are quite similar. As far as the basic properties of the wire are concerned, they are quite adequate in fatigue. Of course, when you put the wire into a concrete beam and you get a crack resulting in some stress concentrations due to that crack, you have other problems to think about.

(Whereupon, the discussion was concluded.)

CURRENT SHORTAGE OF ENGINEERS AND SCIENTISTS

Banquet Address Delivered at 2nd Annual Convention Prestressed Concrete Institute



By
WALTER L. LOWRY, JR.
Head, Civil Engineering Dept.
Clemson College, Clemson, S. C.

Ladies and Gentlemen: When George Ford first asked me to speak to you on this occasion, I was, of course, extremely flattered and immediately accepted before he had a chance to change his mind. My first idea was to attempt to summarize the discussions and papers of the convention, but I decided against this for two reasons: (1) I felt that there was little of merit that I could add to the fine discussions of such outstanding men in the field of prestressed concrete, who have preceded me; and (2) while it is an everyday event for those of us in the field of education to have a group of students sitting helpless before us, it is rather rare that we are able to trap an audience of this size and caliber; and it offers a golden opportunity to discuss a pet problem or situation.

Therefore, I would simply like to mention that in my opinion, and, I am sure, in the opinion of all here tonight, that this convention has been outstanding in every respect up to, but not including, this very minute. All those who have had a part in the planning and organization, as well as those who have participated in the program, are to be congratulated and commended on a job well done.

There have been several references to a certain book on prestressed concrete during the convention. Doctor Lin assured me yesterday that he would not object too strenuously if I also mentioned his book and the dedication therein: "To engineers who, rather than blindly following the codes of practice, seek to apply the laws of nature."

I think that all of us here will agree that there are many aspects of prestressed concrete, both of a theoretical and practical nature, which need further investigation, if the use of this relatively new technique is to continue to expand and take its proper place in the construction industry. To investigate these aspects — to constantly improve techniques in the field — will require men of competence and imagination, thoroughly trained in the basic engineering sciences.

At the same time, we must realize that, as important as this field is at present, it is but one of many, many new fields in the rapidly changing technology of our way of life that will also require an ever-increasing number of men trained in mathematics and

basic sciences, who can tackle new problems and come up with solutions which will meet all challenges, because they are applications of fundamental laws and not the result of the blind following of that well-trodden path of least resistance, the codes of practice.

All of which brings me to my subject for tonight — the shortage of engineers and scientists.

Much has been written recently on this current scarcity of scientists and engineers; indeed, thanks to eager recruiting by employee-hungry companies, it has become one of the best advertised shortages of our time. The finger of blame has been pointed at many different groups but so far, unhappily, the diagnoses have been fine but the remedies few and ineffectual.

In order to arouse your interest — for our history clearly shows that it is only when we are aroused that we take whatever action is necessary — I would like to make certain comparisons between what is happening in the field of education in this country and what is going on in Russia.

Before any discussion of the U.S.S.R., I would first like to emphasize that I hold no brief for the Russian way of life and secondly to present the source of my information. Data for this talk was obtained from publications of the Scientific Manpower Commission, the Russian Research Center of Harvard University, the American Association for the Advancement of Science, and from recent articles in the New York Times, the Saturday Evening Post, and the recent article in Life Magazine by Doctor James R. Killian, President of Massachusetts Institute of Technology. I am sure many of you have read it and those of you who have not, I recommend it as a must.

I would like to trace for you the system that produces the young technical graduate of a Russian University. At the age of three, he enters kindergarten where he is taught the elements of reading to prepare him for the elementary school. In a seven-year elementary school — which, incidentally, every Soviet youngster is required to attend and, if present plans are followed, within a year or two a high-school education will be made mandatory — the student follows a curriculum of which 32 per cent is algebra, geometry, physics, and chemistry. The high schools are preparation for higher education. In these schools there are no electives and over 40 per cent of study time is devoted to mathematics and science. The consensus of opinion of American students of the Russian educational system is that a 17-year-old graduate of the Soviet high school compares very favorably with his American counterpart.

The only test for a student entering the University is merit. Practically every good student is entitled to a government stipend to meet his living expenses in college. This stipend varies considerably according to the state's need for a given profession.

At the present time, the engineering student gets 500 rubles per month — $125.00 at the legal exchange rate — whereas a student of fine arts receives less than $75.00. Once in college — and the courses in science and engineering last five and a half years — the student is subjected to a very rigid schedule which may include as many as 46 hours per week of compulsory classes. The engineering curricula include very little study outside of the purely technical, and the last three years of training are very specialized. Various authorities have disagreed as to the quality of the graduates, but all agree that the level is at least as high as our Master's Degree, and it may approach that of the American Ph. D.

I think that what I have said will give you a good idea of the quality of the engineering and scientific graduate in Russia. Now, let us consider quantity.

There are at present in the U. S. S. R. some 900 technical colleges and 33 universities that are turning out a flood of technically trained people. From a college enrollment of 916,000 in 1953, the Russians produced 43,0000 engineers; in 1954, 53,-000 and in 1955, well over 60,000. This means, of course, that their total college enrollment is increasing at a rapid rate from 916,000 in 1953 to an estimated 2,000,000 in 1956-57.

These figures take on added significance when we compare them to the figures for our own colleges and universities. Soon after World War II many veterans entered engineering colleges and in 1949-50, we had a peak number of engineering graduates of 52,732, representing 12.1 per cent of all college graduates of that year. Then, somehow, the rumor got about that the technical market would soon be flooded. For whatever reason, the high school graduates kept away from engineering with a resulting decline both in actual numbers graduated — from 52,732 in 1949-50 to 22,-236 in 1953-54 and in percentage of engineers to the total college graduates from 12.1 per cent to 7.6 per cent for those years. Although the upswing has begun, it is almost of a too-little and too-late variety. The estimate for 1955-56 is 23,000, when industry estimates an immediate crying need for 50,000 and places the number at somewhere between 60,000 and 70,000 necessary.

This means that besides insisting on higher standards, the Soviets are presently graduating three times more engineers and scientists from their universities than we are from ours.

Instead of our complacent theory that the Soviets "don't have the know-how," perhaps we should prepare ourselves to hear the Russians saying, before too long, that "the United States just doesn't have the know-how." The theory that they have relied on spies and German scientists to build their Badgers and Bisons, A-bombs and H-bombs and the rest is pure, self-deceiving, complacent, nonsense. Foreigners, including spies, have probably contributed less to the Soviet military-industrial successes of the last eight years than the brilliant foreign scientists who fled from Adolf Hitler contributed to our

own Manhattan District Project. Furthermore, the spy-and-German-scientist theory wholly blurs and covers up the meaning of the very ominous military-industrial successes that the Soviets have now achieved.

These successes mean that the Soviet industrial complex is not only brutally massive, but also technologically rich. You simply cannot build A-bombs and H-bombs, Bisons or Badgers without a great array of the most technically advanced supporting industries, such as electronics, and without full mastery of all the most difficult specialized techniques, such as high-temperative metallurgy.

In some cases, indeed, we now know that Soviet technology is actually ahead of our own. For instance, the Bison engine, which develops twice the power of the B-52 engine, showed that Soviet jet-engine design had about two years' lead on United States jet-engine design when the Bison first took the air. Still more interesting was the report of the American automation mission which went to Russia to inspect their applications of the newest production methods.

The mission was composed of hardheaded American engineers, such as Nevine L. Bean, top technical man in the Ford Motor Company's automatic-transmission plant, who could never have been fooled by papier-mache' assembly lines. They saw what they asked to see, and their request list of factories to be visited was as fully representative as the American intelligence could make it.

Bean returned from his visit much impressed. One of their initial experiments with automation, a ball-bearing production line, struck Bean as just a little better than anything of the same sort in America. His final verdict was that the Soviets are coming up into a very high technical era pretty frightening to think about.

I would like to stress one particular aspect of our present educational system with you. In this country it is becoming increasingly difficult, if not almost impossible, to interest young people in a teaching career for our elementary schools, our high schools, or our colleges. Particularly this is true concerning mathematics and science teachers for our high schools—in fact, mathematics courses were dropped in 1500 high schools in the past year alone—and for engineering staff members for our colleges. In the last four years the number of college graduates trained for high school teaching of mathematics has fallen 56 per cent. Many who might go into teaching have found more profitable jobs in government or private industry.

Within the next 10 years, the high school enrollments will increase from the present 7,000,000 to 10,000,000. The threat to the high school science program—and it is at this level that the future college engineers and scientists receive their initial training—is ominous.

How then are the Soviets finding their mathematics and science teachers for their high schools and their engineering professors for their universities? The answer is ridiculously simple and I would like to answer it by another illustration. A colonel in the Russian Army draws about $1,000.00 a month which is approximately equal to what colonels in our army receive. Professors in Russia get between $1,500.00 and $3,000.00 per month, and, in addition, get special bonuses and supplementary salaries for secondary jobs such as serving on special literary or scientific commissions and many other fringe benefits. As a result, a recent survey showed that there were eight qualified applicants for each vacancy in the teaching field in Russia. I leave it to your imagination and, particularly, those in the educational field, to consider eight applicants of any sort.

This talk might be interpreted as advocating that we should all, for survival's sake, study science or engineering, but such is not my intention. While I firmly believe that we should all be aware of the developments in science and understand how such developments affect our lives, we must not allow the pressure for scientists and engineers to obscure the need for first-rate talent in other fields—that we should not engage in a numbers race with the Soviets. Rather, we can win through enterprise, efficiency, unexcelled quality, fully mobilized and motivated intellectual resources.

To repeat, much has been written and said about the cause for the current scarcity of scientists and engineers; but rather than the negative approach of placing blame, we should be thinking of a positive program for the solution of the problem in the future. Doctor Killian, in his artcile, advocated four positive moves to meet our shortage which, I think, will stand repetition and emphasis.

First, the nation should establish several thousand more scholarships to help gifted but needy high school graduates go to college. Only a third of the high school graduates standing in the top third of their classes are presently going on to get college degrees and thus qualify for the professions. The other two-thirds of these top high school graduates—and they number some 200,000 annually—apparently do not want to go to college or they cannot afford to go—but, the nation cannot afford for them not to afford or want to go.

The scholarship funds provided by colleges, corporations, states, and foundations now total between $50,000,000 and $60,000,000 annually. Doctor Killian thinks that increasing this by 50 per cent now, doubling the total in two or three years, and quadrupling it in five would begin to bring our student aid resources to the level our growing needs require.

This increase should be financed by private funds as, for example, through a concerted effort by foundations and corporations, but if this goal cannot be reached by private funds, then a short-term program financed by the Federal Government should be instigated. Such a program would highlight the need for the lifting of sights in our high schools and for more adequate oppor-

tunities and rewards for gifted students. It would help to encourage better and more advanced teaching of science in high schools and thus it would enable students of ability to cover more ground in college.

Second on Doctor Killian's list of remedial measures would be a concerted effort to revitalize the teaching of science and mathematics in the secondary schools. One suggestion is that the best men in a particular field be assembled in a special study group to plan a whole new approach to elementary education in that field. Let them set new goals, design new courses, visualize fresh experiments and imaginative demonstrations. Let these plans be reduced to reality by a working group which would design and direct the making of professionally executed movie shorts—such as a 20-minute length short. Each would have the necessary manuals and texts. In this way we could capitalize on the resources of visual technology to give to the secondary school teacher a fresh and powerful teaching aid and a new reach.

Third on the list of needed actions is a national forum like the recent White House Conference on Education to throw a spotlight on ways whereby the nation can educate more and better scientists and engineers. As an example, many industrialists and educators feel that engineering education is in need of revitalization. Such questions as: "Should it contain more basic science?" and "Do we need a new breed of engineer, competent to design and manage great engineering systems and skillful to handle the human problems involved?" need to be examined and discussed in a national forum where they can be illuminated for the public at large.

Fourth, Doctor Killian suggests that we formulate the outlines of a grand strategy for maintaining our technological advance in the United States. The President's Air Policy Commission exerted great influence through its report "Survival in the Air Age." Scientists and engineers advisory to the government are organizing a comparably thorough report to chart our technological progress in the years ahead. Along this line, President Eisenhower announced just a month ago today the membership of a committee of 33 on Education Beyond the High School. In addition, he has appointed a National Committee for the Development of Scientists and Engineers. Chairman and Vice-Chairman of the 19-member committee are, respectively, Howard L. Bevis, President of Ohio State University and Eric A. Walker, Dean of Engineering and Architecture at Pennsylvania State University. This committee's responsibilities, as outlined in a letter from the President to Mr. Bevis, are (1) to identify the problems connected with the development of more highly qualified scientists and engineers; (2) to enlist the cooperation of all interested individuals and groups in analyzing the problems and developing programs; (3) to make available information on effective ways of overcoming the obstacles to training this type of manpower; (4) to publicize the problem and possible solutions; and (5) to provide the President with a progress report, from time to time.

In addition to these committees, there are many, many other bright spots on the horizon. Time will permit mention of only one or two:

(1) Elementary and secondary schools are at last recognizing that their primary purpose is academic and that practical skills can be learned elsewhere;

(2) Special recognition is being given to outstanding students, for example, at Notre Dame, the top 10 per cent of the liberal-arts students are relieved of routine requirements after their freshman year and are assigned to special seminars. Thus, the extra-bright are encouraged to get their first degree in three years and then go on to graduate work at the University's expense.

(3) Corporations and foundations are stepping up contributions to educational institutions and are encouraging their employees to do likewise. Examples of this are the Ford Foundation's recent grants, the General Electric's programs of matching gifts of its employees to their colleges up to $1,000.00 per individual per year, B. F. Goodrich's recently announced three-point annual program of seven four-year scholarships for high school graduates, payment of 50 per cent of tuition and laboratory fees for any employee taking a course related to his job, and matching gifts of up to $500.00 per year that an employee makes to any school, the Westinghouse program, General Motors program, Rockefeller Foundation, and others too numerous to mention.

I'm afraid that this extra course at this banquet — I won't call it a dessert since a dessert is generally short and sweet — has been pretty heavy, but I hope not too indigestible. I have violated the fundamental rules of a good speech — a good beginning and a good ending as close together as possible. I will, therefore, close with this one last thought.

Each year this country is pouring vast sums of money into research and development in the search for new materials and resources. Isn't it time for us to combine our efforts to see that proper attention is given to the number one resource of our country—the youth of America?

● Memo to advertising MANAGERS

WANT a salesman that reaches top management personnel in architectural and engineering offices . . . in state and federal engineering departments, and in prestressed concrete product plants?

NEED a market in one of the fastest growing new industries in the country with an unlimited potential?

THEN advertise your materials, equipment or services in the PCI Journal, the only publication covering the B-I-G prestressed concrete industry. The JOURNAL is the official technical publication of the Prestressed Concrete Institute, a national organization dedicated to the advancement of this booming new industry. It delivers concentrated buying power . . . circulation where it counts.

JOURNAL PUBLICATION OFFICE

3132 N. E. Ninth Street
Fort Lauderdale, Florida

MEMBERSHIP QUALIFICATIONS

SCHEDULE OF DUES

Anyone engaged in the production of prestressed concrete, or in the production of materials or equipment allied to the prestressing industry; licensed architects and engineers; engineers and architects in training, and students enrolled in accredited schools of architecture and engineering, are invited to apply for membership in the Prestressed Concrete Institute. Membership classifications have been established as follows:

ACTIVE — **$250.00**
Organizations and individuals actively engaged in the production of prestressed concrete products.
Annual membership Dues

ASSOCIATE — **$250.00**
Organizations and individuals engaged in the production of materials and equipment allied to the prestressed concrete industry.
Annual membership Dues

PROFESSIONAL — **$ 25.00**
Limited to registered architects and engineers.
Annual membership Dues

JUNIOR — **$ 15.00**
Limited to architects and engineers in training.
Annual membership Dues

STUDENT — **$ 5.00**
Limited to students of accredited architectural and engineering schools.
Annual membership Dues

APPLICATION FOR MEMBERSHIP IN PRESTRESSED CONCRETE INSTITUTE
(A non-profit corporation)

1. Name and address of company applying for membership:

_____

_____

_____

2. Name of person to represent company in Institute affairs:

_____

3. Type of membership applied for:

☐ Active (manufacturer)

I have done the following types of prestressing:

☐ Post-tensioning
☐ Pre-tensioning
☐ Combination of both of above
☐ Precasting
☐ On-the-job post-tensioning
☐ On-the-job pre-tensioning
☐ Design

☐ Associate (related business)

I am in the business of_____

_____

_____

☐ Professional (registered architect or engineer)
I hold certificate No._____of the State of_____.

☐ Junior (limited to architects and engineers in training)

☐ Student (limited to students of accredited schools of architecture and engineering)

Name of School_____

I understand the financial arrangements and am not to make any payment until approval of this application for membership.

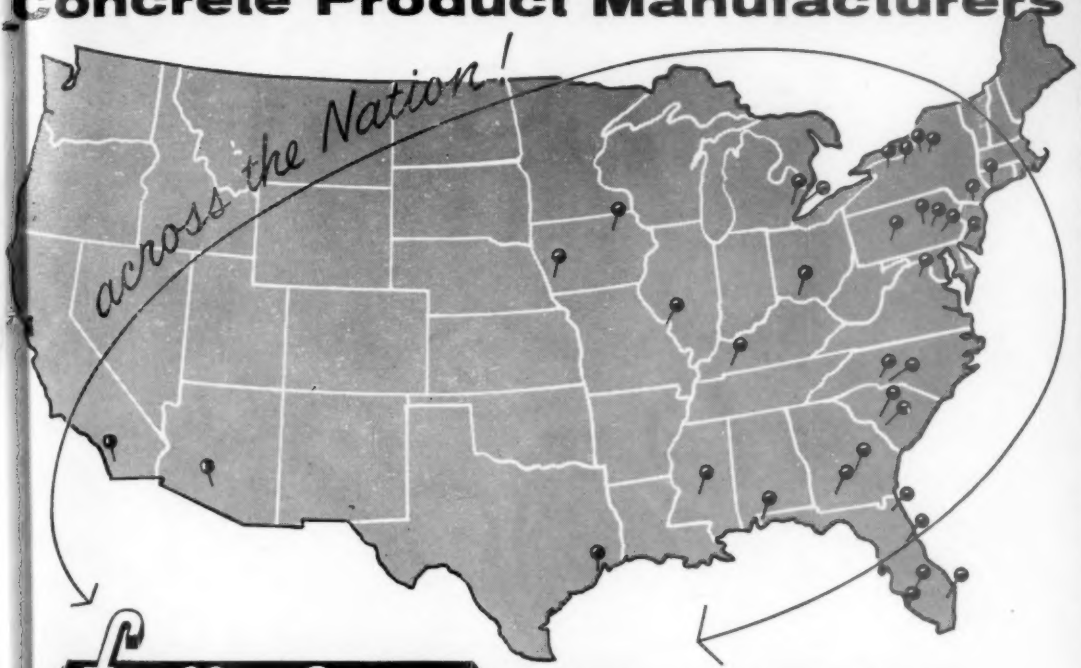
Signed:_____

By_____

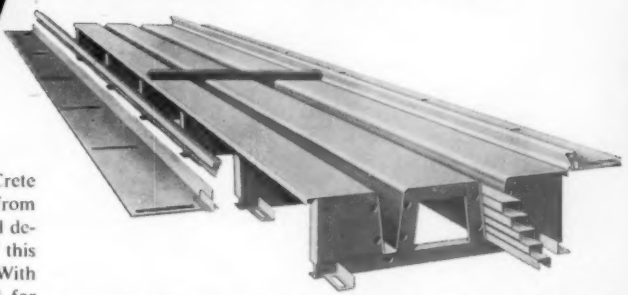
(cut along this line)

PRINT IN BINDING

The choice of Prestressed Concrete Product Manufacturers



FORM-CRETE® STEEL FORMS



The immediate acceptance and purchase of Form-Crete Steel Forms by prestressed concrete manufacturers from coast to coast provides ample assurance of the sound design, practical rugged construction and economy of this new innovation in the prestressed concrete field. With the addition of a line of I-beam forms engineered for casting all standard specified sizes of prestressed concrete I-beams we offer the most complete line of coordinated interchangeable forms and accessories obtainable anywhere. A new revised catalog is available soon—write for your copy today.

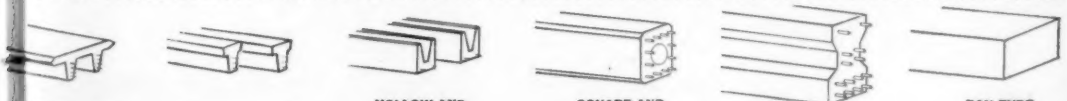
FORM-CRETE DOUBLE-T ALL-STEEL CASTING FORMS—*By utilizing a variety of gout forms and stem heights an ingenious manufacturer can easily surpass the 37 individual prestressed concrete castings that we conservatively estimate may be produced from this one all-steel form assembly.*

SEND FOR **FORM-CRETE BULLETIN 100-A**

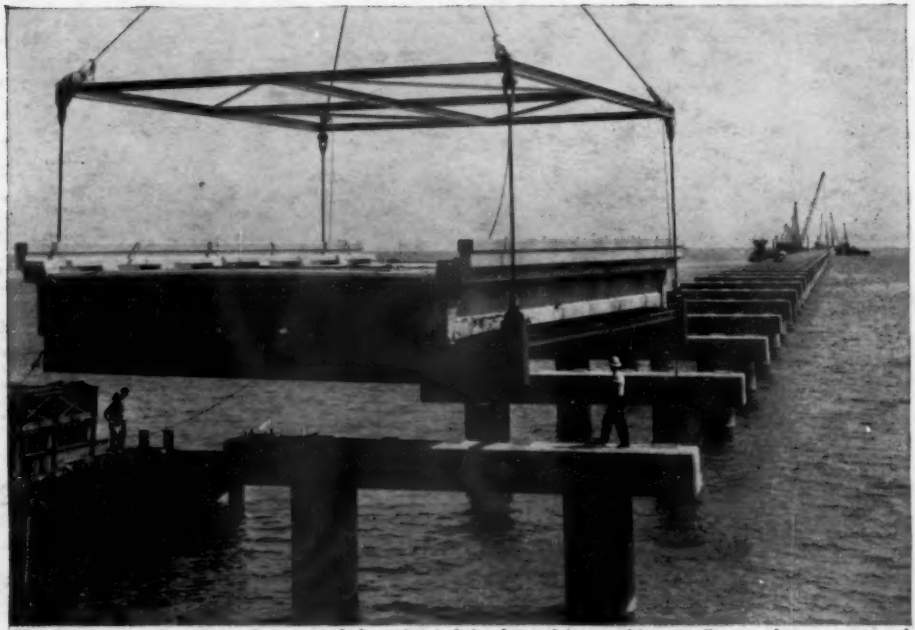
PF-1

FOOD MACHINERY
AND CHEMICAL CORPORATION
FLORIDA DIVISION
LAKELAND, FLORIDA

FORM-CRETE STEEL FORMS FOR CASTING REINFORCED OR PRESTRESSED CONCRETE



SOLID "T" SLABS | SINGLE "T" JOISTS | HOLLOW AND SOLID LINTELS | SQUARE AND OCTAGONAL PILING | BRIDGE BEAMS | PAN TYPE BRIDGE DECKS



This 180-ton precast, prestressed concrete deck section is being lowered into position on pile caps that are supported on hollow 54" prestressed piles. There are 2,235 of these deck spans on the 24 mile causeway across Lake Pontchartrain near New Orleans.

Why They Used Plastiment in the World's Longest Highway Bridge...

Efficient single unit beam-and-deck design and economical mass production are outstanding features of this 24 mile causeway across Lake Pontchartrain. Each day eight 56 foot deck-spans are cast in a single bed containing pretensioned wires.

Plastiment Concrete Densifier is being used for these reasons:

UNIFORM HIGH QUALITY YEAR AROUND — During cooler weather the minimum quantity of Plastiment is used for highest structural quality. More Plastiment is added in the hot summer to keep water content and setting time of concrete constant. Cracking and crazing are entirely eliminated.

HIGH EARLY STRENGTH FOR PRESTRESSING — Initial retardation allows time for placement and consolidation of the entire unit (89 cubic yards) even in the hottest weather. After this retardation, Plastiment concrete gains strength rapidly. Steam curing for 15 hours accelerates this gain. At three days, strength averages well above 4000 psi.

ECONOMY — An economical 6½ bag concrete mix, with varying proportions of Plastiment, results in uniform high structural quality well above specifications. Ease of placement and compaction, a characteristic of low slump Plastiment concrete, reduces placing costs.

Engineers: Palmer and Baker, Inc.
Mobile, Alabama

Contractor: Louisiana Bridge Company —
A joint venture:
Brown & Root, Inc. — Houston, Texas
T. L. James Co., Inc. — Ruston, Louisiana

Write today for information on producing uniform concrete at all temperatures.

Sika **PLASTIMENT®**
CONCRETE DENSIFIER
SIKA Chemical Corporation, Passaic, N. J.

DISTRICT OFFICES: Boston • Chicago • Dallas • Philadelphia Pittsburgh • Salt Lake City • Washington, D. C. • Montreal, Vancouver, Canada • Panama City, Panama • Dealers in Principal Cities